DTIC FILE COPY

4

# Special Report 89-2

January 1989

US Army Corps of Engineers
Cold Regions Research & Engineering Laboratory

# *Response of pavement to freeze-thaw cycles*

## *Lebanon, New Hampshire, Regional Airport*

Wendy L. Allen, William F. Quinn, Donald Keller and Robert A. Eaton





Prepared for
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL AVIATION ADMINISTRATION

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE

# REPORT DOCUMENTATION PAGE

Form Approved
OMB NO. 0704-0188
Exp. Date: Jun 30, 1986

| | |
|---|---|
| 1a. REPORT SECURITY CLASSIFICATION<br>Unclassified | 1B. RESTRICTIVE MARKINGS |
| 2a. SECURITY CLASSIFICATION AUTHORITY<br>2b. DECLASSIFICATION/DOWNGRADING SCHEDULE | 3. DISTRIBUTION/AVAILABILITY OF REPORT<br>Approved for public release; distribution is unlimited. |
| 4. PERFORMING ORGANIZATION REPORT NUMBER(S)<br>Special Report 89-2 | 5. MONITORING ORGANIZATION REPORT NUMBER(S)<br>DOT/FAA/PS-89/1 |
| 6a. NAME OF PERFORMING ORGANIZATION<br>U.S. Army Cold Regions Research and Engineering Laboratory<br>6b. OFFICE SYMBOL (if applicable)<br>CECRL | 7a. NAME OF MONITORING ORGANIZATION<br>Federal Aviation Administration<br>Program Engineering and Maintenance Service |
| 6c. ADDRESS (City, State, and ZIP Code)<br>72 Lyme Road<br>Hanover, N.H. 03755-1290 | 7b. ADDRESS (City, State, and ZIP Code)<br>Washington, D.C. 20591 |
| 8a. NAME OF FUNDING/SPONSORING ORGANIZATION<br>8b. OFFICE SYMBOL (if applicable) | 9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER<br>DOT-FA79WA1-059 |
| 8c. ADDRESS (City, State, and ZIP Code) | 10. SOURCE OF FUNDING NUMBERS<br>PROGRAM ELEMENT NO. 6.27.30A<br>PROJECT NO. 4A7627 30AT42<br>TASK NO. D<br>WORK UNIT ACCESSION NO. 002 |

11. TITLE (Include Security Classification)

Response of Pavement to Freeze-Thaw Cycles: Lebanon, New Hampshire, Regional Airport

12. PERSONAL AUTHOR(S)
Allen, Wendy L.; Quinn, William F.; Keller, Donald and Eaton, Robert A.

| 13a. TYPE OF REPORT | 13b. TIME COVERED | 14. DATE OF REPORT (Year, Month, Day) | 15. PAGE COUNT |
|---|---|---|---|
| | FROM ______ TO ______ | January 1989 | 35 |

16. SUPPLEMENTARY NOTATION

| 17. COSATI CODES | | | 18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number) |
|---|---|---|---|
| FIELD | GROUP | SUB-GROUP | Asphalt concrete; Nondestructive testing; Resilient stiffness; Frost heaving; Pavements |

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

In 1978 reconstruction was begun on the runway of the Lebanon Regional Airport, Lebanon, New Hampshire. The runway had experienced severe differential frost heaving and cracking during the previous three winters, which had resulted in closure of the facility during periods of extreme roughness. At the request of the Federal Aviation Administration and in conjunction with the reconstruction, CRREL undertook to study the newly constructed pavements to investigate the relationships between weakening of the pavements, frost heave of the pavement surfaces and the position of the freezing front. Temperature sensors were placed within the newly constructed pavement sections, and during the winters of 1979, 1980 and 1982 temperature data were recorded, and level surveys and repeated plate bearing tests were performed in order to provide data for the investigation. The three pavement sections were constructed to investigate the effect of section thickness on the level of frost protection provided. The sections consisted of 4 in. of asphalt concrete, 6 in. of crushed gravel and 22, 30 and 38in. of well-graded sand subbase material. The 48-in. section provided the highest level of frost protection to the subgrade. However, all three pavement sections maintained resilient stiffness values during the spring thaw period on the order of two to three times that of the pavement before reconstruction. Also, frost heave in all sections was reduced to levels that would not cause difficulty for aircraft using the facility. (SDU)

| | |
|---|---|
| 20. DISTRIBUTION/AVAILABILITY OF ABSTRACT<br>☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS | 21. ABSTRACT SECURITY CLASSIFICATION<br>Unclassified |
| 22a. NAME OF RESPONSIBLE INDIVIDUAL<br>Wendy L. Allen | 22b. TELEPHONE (Include Area Code) 603-646-4100<br>22c. OFFICE SYMBOL CECRL-EC |

DD FORM 1473, 84 MAR — 83 APR edition may be used until exhausted. All other editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE

UNCLASSIFIED

## PREFACE

This report was prepared by Wendy L. Allen, Research Civil Engineer, William F. Quinn, Chief, and Donald Keller, formerly a Civil Engineering Technician, of the Civil and Geotechnical Engineering Research Branch, Experimental Engineering Division, and Robert A. Eaton, Civil Engineer, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory.

Primary funding for this study was provided by the Department of Transportation, Federal Aviation Administration, under Interagency Agreement DOT-FA79WAI-059, 15 July 1979. The purpose of the study was to collect data at the Lebanon, New Hampshire, Regional Airport on the relationship between thaw weakening of pavement, as determined by nondestructive testing, and the position of the freezing front. Secondary funding was provided by the Office, Chief of Engineers under DA Project 4A762730AT42, *Design, Construction and Operations Technology for Cold Regions*, Task D, Work Unit 002, *Use of Frost Susceptible Soil in Roads and Airfields*.

The authors recognize North Smith who was the initial CRREL project engineer on the study and was responsible for selection of the location of observation points, the installation of temperature sensors, and the observation and analysis of data obtained during the preconstruction, construction, and the 1978–79 post-construction seasons. The authors would also like to recognize James Bates, formerly a CRREL engineer, who conducted the observation and analysis program during the 1979–80 winter season, and Dr. Richard Berg of CRREL who technically reviewed this report. Mr. Bates also conducted the statistical study to determine the relationship between pavement temperature and stiffness.

## CONTENTS



## ILLUSTRATIONS

## TABLES

# Response of Pavement to Freeze-Thaw Cycles
## Lebanon, New Hampshire, Regional Airport

WENDY L. ALLEN, WILLIAM F. QUINN,
DONALD KELLER, AND ROBERT A. EATON

## INTRODUCTION

Runway 18/36 at the Lebanon Regional Airport, Lebanon, New Hampshire, experienced severe differential frost heaving and cracking during the winters of 1975, 1976, and 1977, which caused the runway to be closed for safety reasons. Photographs in the local newspaper showing 1- to 2-in.-wide cracks next to the airport manager's car aroused public sentiment to support reconstruction of the runway. Funding was obtained and plans drawn for reconstruction in the summer of 1978.

At the request of the Federal Aviation Administration, and in conjunction with the 1978 reconstruction of runway 18/36, CRREL instrumented the reconstructed pavement sections and collected data to investigate the relationships between the weakening of pavements (as determined by nondestructive testing), frost heave of pavement surfaces, and the position of the freezing front.

Field observations over the three-year test period were scheduled so that seasonal variations in pavement strength could be assessed. Measurements taken included pavement temperature, temperature of soils at depth, pavement surface deflections as measured by the repeated plate bearing (RPB) test apparatus, and pavement surface elevations to evaluate frost heave. Measurements of soil moisture within the pavement structure were not included in the scope of this work.

The three winter seasons studied were 1978–79, 1979–80, and 1981–82, with freezing indices of 1205, 975 and 1284°F-days, respectively. The air freezing indices are based on air temperatures obtained at the Lebanon airport. The mean freezing index for this area is 1060°F-days, based on temperature data obtained at the Dartmouth College meteorological station, about five miles north of the airport site. The design air freezing index is 1820°F-days and is based on the average of the three coldest years in 30 years at the Dartmouth College Station (Gilman 1964).

A plan of the airfield for the Lebanon Regional Airport is shown in Figure 1.

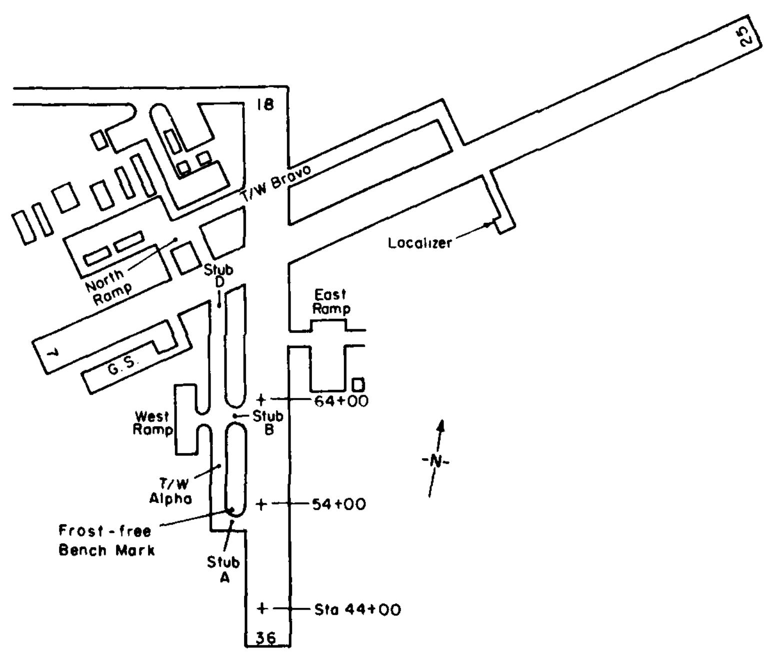
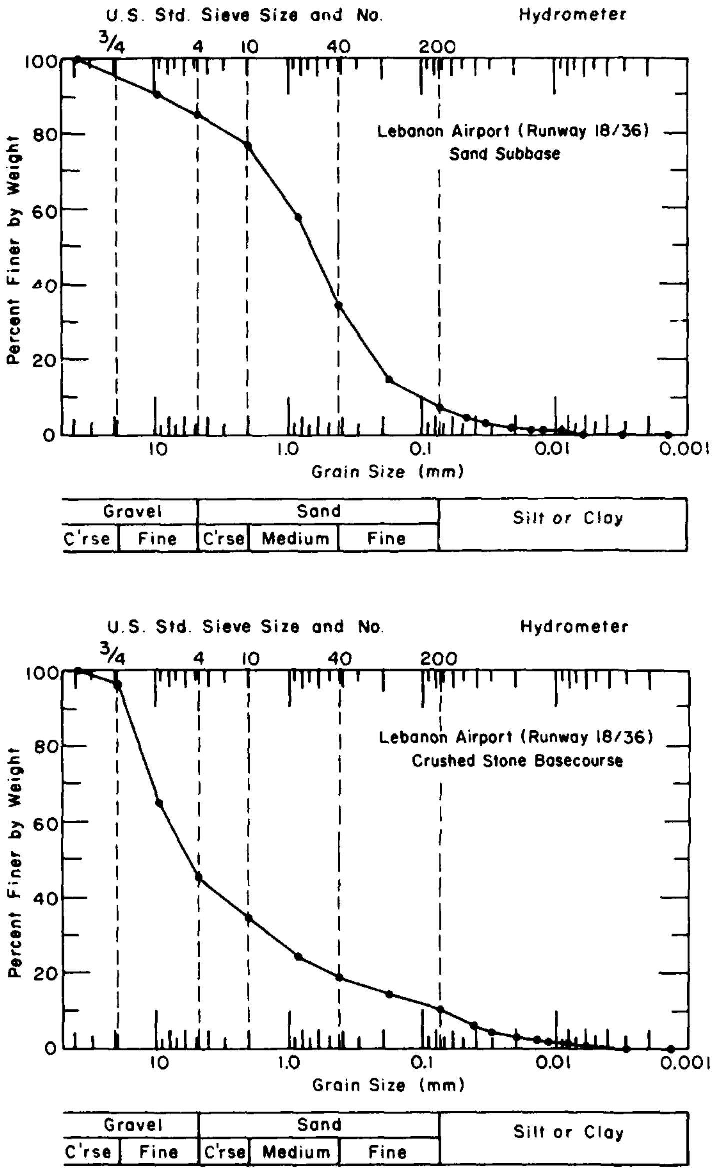



*Figure 1. Layout of Lebanon Regional Airport.*




*a. Subbase sand.*

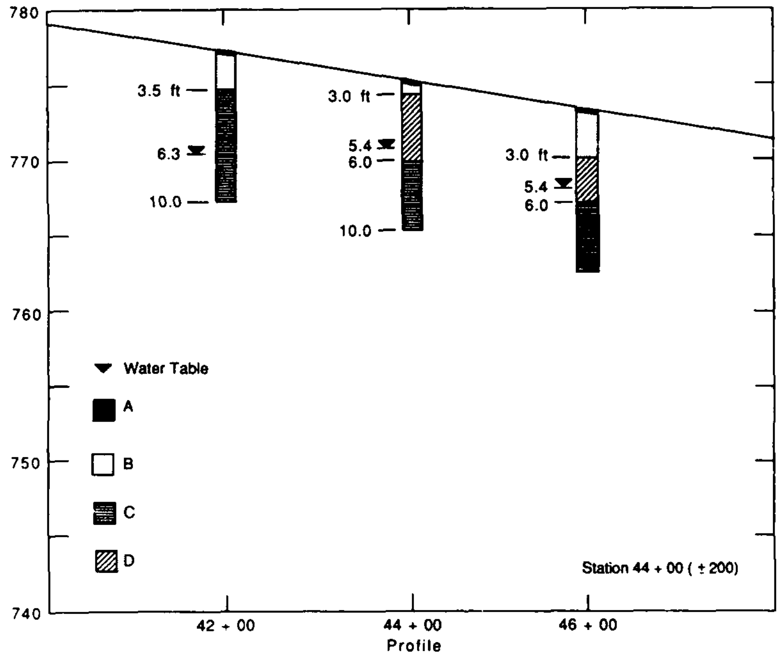
*b. Crushed stone base.*

*Figure 2. Grain size distribution of subbase sand and crushed stone base.*

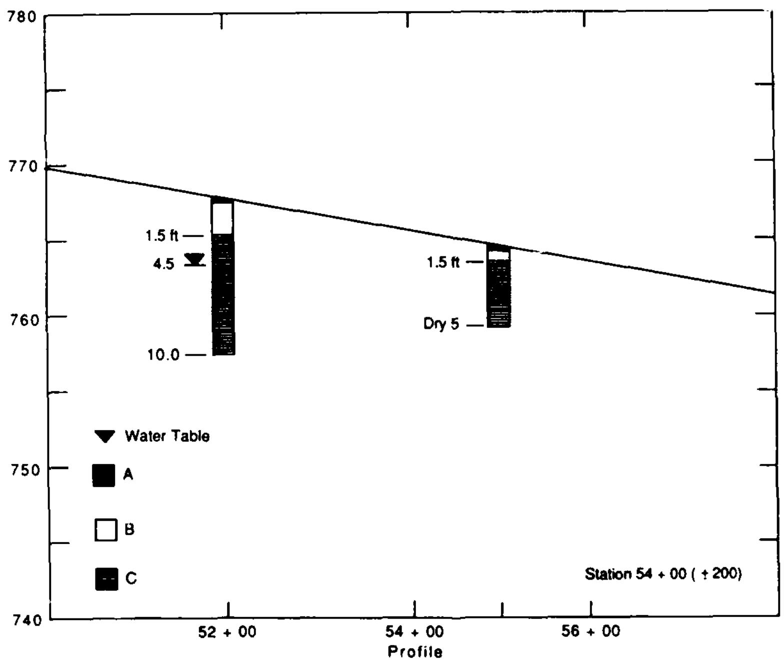
## RUNWAY PAVEMENT STRUCTURE

The work on runway 18/36 involved reconstruction of the runway pavement only; the existing shoulders were not reconstructed. The architect-engineer's design of the reconstructed pavement was based on a design aircraft with a 60,000-lb dual wheel load and the limited subgrade frost penetration design procedure (Berg and Johnson 1983). These criteria resulted in a design pavement structure of 4 in. of asphalt concrete pavement (a 1.5-in. wearing course and a 2.5-in. binder course), 6 in. of crushed gravel and a minimum of 22 in. of subbase material.

The subbase course consists of a well-graded sand with a trace of gravel having a maximum size of 4-in. The sand subbase has about 8% material finer than the no. 200 sieve and about 2% finer than the 0.02-mm grain size. The crushed base course has about 11% material finer than the no. 200 sieve and about 4% finer than the 0.02-mm grain size. Subbase and base course grain size distribution curves are shown in Figures 2a and 2b. The Corps of Engineers classifies

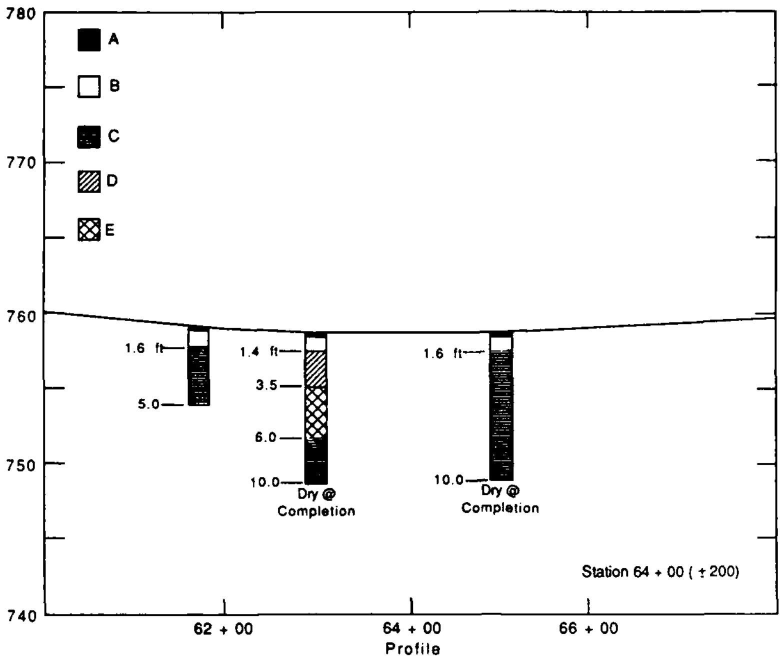



*a. Station 44+00 and vicinity.*




*b. Station 54+00 and vicinity.*

***Figure 3. Preconstruction boring logs (A: asphalt concrete pavement, 3-in. typical; B: fine to coarse gravel and fine to coarse sand, trace of silt; C: brown moist silt, little fine to coarse gravel, trace of fine to coarse sand; D: gray, moist silt, little fine to coarse sand, trace of fine to coarse gravel; E: silt).***




*c. Station 64+00 and vicinity.*

*Figure 3 (cont'd). Preconstruction boring logs.*

most soils having 3% or less material finer than the 0.02-mm grain size as non-frost-susceptible.

Preconstruction boring logs obtained in the vicinity of station 44+00 are shown in Figure 3a, station 54+00 in Figure 3b, and station 64+00 in Figure 3c. The logs indicate gravel or sand underlain by a silty subgrade, as classified by the soils testing firm retained on the contract. This silty material could be expected to range between low and very high frost susceptibility based on the Corps of Engineers frost susceptibility classification system.

Removal of material deemed to be unsuitable at the time of construction excavation resulted in a reconstructed pavement of variable subbase thickness. The subbase thickness varied as follows: 22 in. at station 44+00, 30 in. at station 54+00, and 38 in. at station 64+00. These three runway stations were selected for the airport observational program.

## PRECONSTRUCTION TESTING

In order to establish some baseline information on the pavement stiffness of runway 18/36 prior to reconstruction, 27 RPB tests were conducted on the old pavement surface in late March and early April 1978 when thawing of the pavement and subgrade had just been completed. In late April 1978, RPB tests were conducted at four locations previously tested to assess strength recovery following reconsolidation and partial drainage of excess soil moisture.

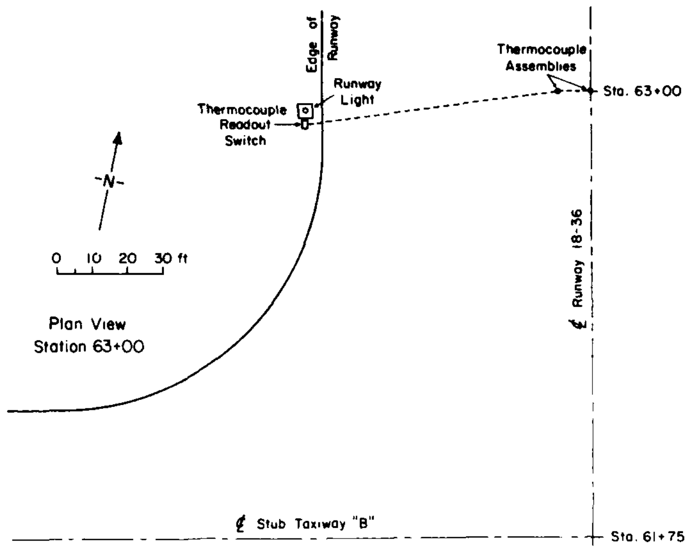
## OBSERVATIONAL PROGRAM

### Thermocouples

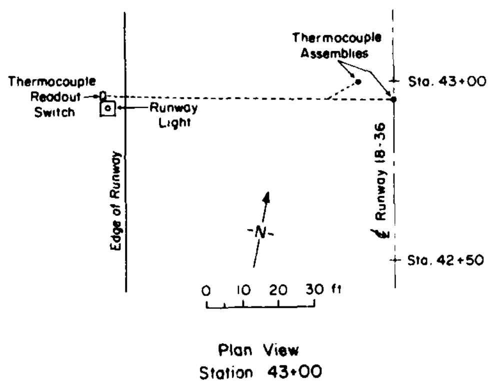
Four strings of copper-constantan thermocouples were installed in the reconstructed pavement to monitor temperatures in the subgrade, subbase, base and pavement. Two strings were located at station 63+00, one at the runway centerline and the other 10 ft left of centerline. The other two strings were located at station 43+00, at the centerline and about 13 ft left of center. The locations are shown in plan in Figures 4a and 4b. Cross-sectional views showing thermocouple locations are shown in Figure 5a for station 63+00 and in Figure 5b for station 43+00.

### Level surveys

The pavement's cross-sectional frost heave was monitored across the entire pavement width at stations 44+00, 54+00 and 64+00, using 21




*a. Station 63+00.*




*b. Station 43+00.*

*Figure 4. Location of thermocouple assemblies.*

designated points across the 100-ft-wide runway section and both of the 25-ft-wide shoulders on either side of the runway.

**Nondestructive testing**

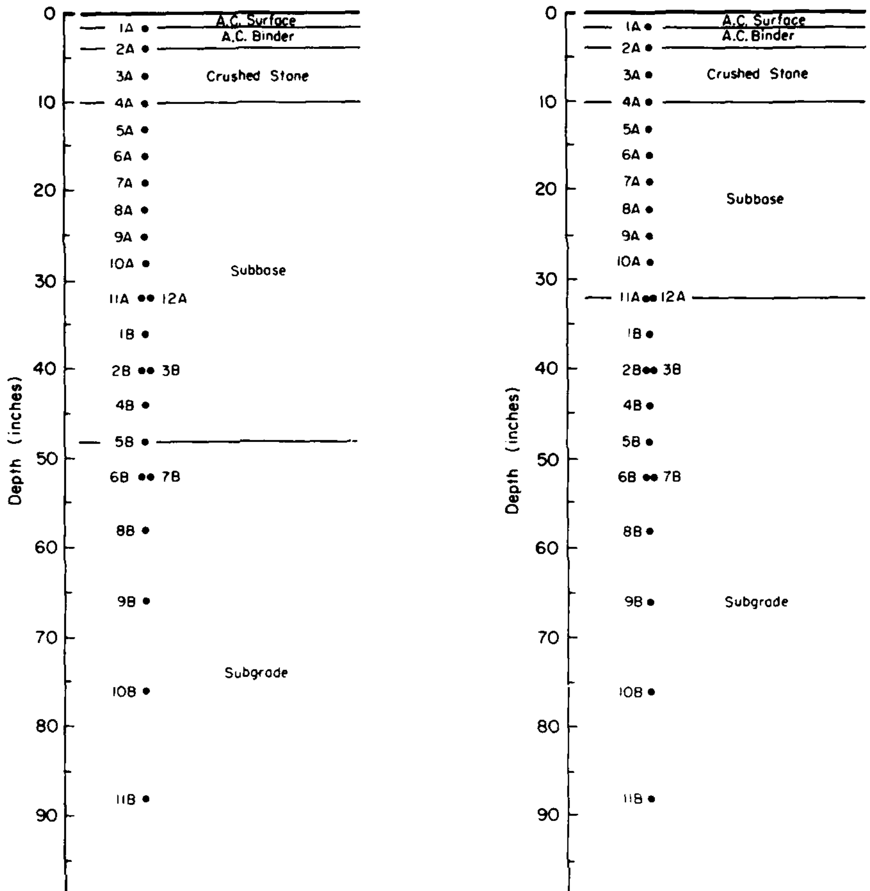
Nondestructive testing was accomplished using a RPB test apparatus. The CRREL RPB test vehicle is a self-contained trailer-mounted pavement testing apparatus that allows field repetitive plate bearing testing of roads and airfields with a minimum of setup and take-down time (Fig. 6). The trailer, which has a gross vehicle weight of 30,000 lb, is 27 ft long, 12 ft high, and 8 ft wide. The design axle load of the trailer is 18,000 lbf so that this vehicle can also be employed to conduct standard Benkelman beam static rebound tests. More detailed information on the apparatus is provided in Appendix A.

The RPB unit can generate successive load pulses in the 1- to 14-kip (1,000- to 14,000-lbf) range at rates of up to 20 repetitions per minute. The profile of a typical series of 9-kip load pulses is shown in Figure 7a; a strip-chart recorder monitors the LVDT and load cell output (Fig. 7b). The vehicle contains all the instruments required to provide a continuous recording of the force transmitted to the pavement surface and the motion of the pavement surface within a 4-ft radius of the circular load plate.

For the tests conducted at the Lebanon airport, a 9-kip load pulse was applied 200 or 500 times to a 12 in.-diam. load plate, resulting in a nominal contact pressure of 80 lb/in.$^2$. Two linear variable differential transformers (LVDTs) were mounted on a reference beam at 90° to each other to record the motion of the load plate. Four additional LVDTs were mounted along the reference beam to monitor the deflection of the pavement along one radius projected outward from the load plate (Fig. 8).

At the time that the RPB measurements were being made, both pavement and air temperatures were also recorded using a thermocouple probe. Throughout the three-year testing program, a total of 44 RPB tests were conducted at locations 10 ft left of the centerline at stations 44+00, 54+00 and 64+00.

The tests were conducted, on the average, on a biweekly basis between October and April. During periods when the frost penetration depth was essentially stable, tests were conducted on a monthly basis. During periods of rapid thaw, generally March, tests were conducted weekly.




a. Station 63+00. b. Station 43+00.

*Figure 5. Sectional view, thermocouple assemblies.*

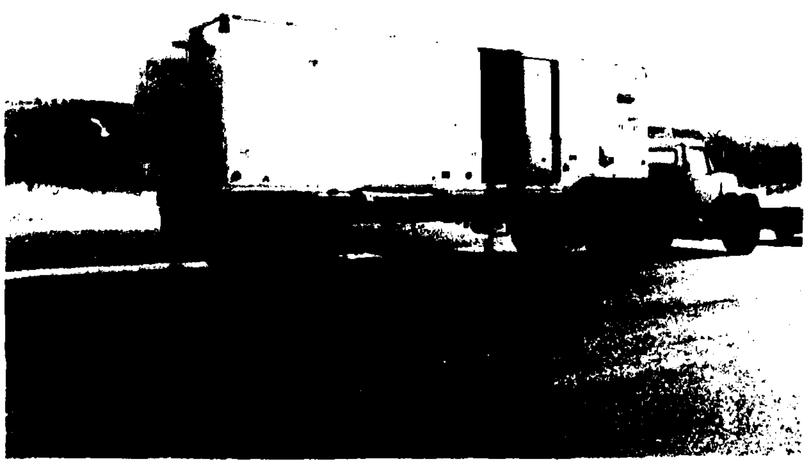
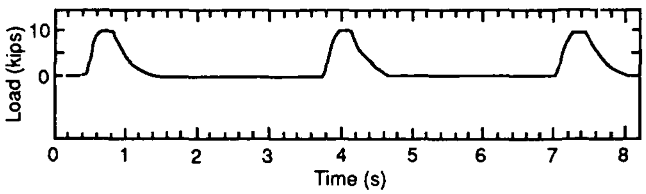


*Figure 6. Repeated plate bearing test van.*




*a. Typical load pulses generated by the RPB apparatus.*

*b. RPB strip chart recorder.*

*Figure 7. Typical load pulses generated by the RPB apparatus and the RPB strip chart recorder.*

*Figure 8. RPB reference beam.*

**Table 1. Resilient stiffness observations—preconstruction.**

| Station | Location | Date | $S_R$ (kips/in.) | Load (lbf) | Pavement temp (°C) | (°F) |
|---|---|---|---|---|---|---|
| 40+00 | * | 3/29/78 | 200 | 8875 | 14.2 | 57.6 |
| 42+00 | 10'L | 3/29/78 | 153 | 8750 | 14.0 | 57.2 |
| | | 4/26/78 | 233 | 8258 | 7.7 | 45.9 |
| 44+00 | 15'R | 3/29/78 | 111 | 8750 | 13.6 | 56.5 |
| 46+00 | 15'R | 3/29/78 | 217 | 8650 | 11.1 | 52.0 |
| | | 4/26/78 | 284 | 8390 | 16.6 | 61.9 |
| 48+00 | 15'R | 3/31/78 | 203 | 8750 | 8.4 | 47.1 |
| 50+10 | 45'L | 3/31/78 | 159 | 8500 | 9.3 | 48.7 |
| | 30'L | 3/31/78 | 211 | 8750 | 9.4 | 48.9 |
| | 15'L | 4/3/78 | 207 | 8650 | 12.5 | 54.5 |
| | * | 3/31/78 | 142 | 8500 | 11.1 | 52.0 |
| | * | 4/3/78 | 212 | 8650 | 9.5 | 49.1 |
| | 15'R | 4/3/78 | 218 | 8700 | 10.0 | 50.0 |
| | 30'R | 3/31/78 | 176 | 8650 | 10.3 | 50.5 |
| | 45'R | 4/3/78 | 250 | 8700 | 11.5 | 52.7 |
| 51+90 | 10'R | 3/31/78 | 109 | 8800 | 8.2 | 46.8 |
| | * | 4/26/78 | 201 | 8792 | 16.9 | 62.4 |
| 58+00 | 10'R | 3/31/78 | 138 | 8700 | 8.8 | 47.8 |
| 61+75 | 15'R | 3/31/78 | 119 | 8600 | 6.2 | 43.2 |
| 63+00 | 45'L | 3/30/78 | 91 | 8700 | 11.3 | 52.3 |
| | 45'L | 4/26/78 | 255 | 8500 | 24.5 | 76.1 |
| | 30'L | 3/30/78 | 162 | 8650 | 11.6 | 52.9 |
| | 15'L | 3/30/78 | 136 | 8750 | 15.6 | 60.1 |
| | * | 3/30/78 | 115 | 8650 | 16.1 | 61.0 |
| | 15'R | 3/30/78 | 169 | 8800 | 9.0 | 48.2 |
| | 30'R | 3/30/78 | 203 | 8875 | 11.6 | 52.9 |
| | 45'R | 3/31/78 | 193 | 9000 | 1.3 | 34.3 |
| 65+00 | 10'L | 3/30/78 | 190 | 9000 | 5.1 | 41.2 |
| 68+10 | 10'L | 3/29/78 | 150 | 9000 | 10.5 | 50.9 |
| 69+10 | 20'R | 3/29/78 | 142 | 8850 | 11.2 | 52.2 |

* Centerline

## TEST RESULTS

### Preconstruction period

The data for both the pre-construction and corresponding post-construction RPB tests are given in Table 1.

The resilient stiffness $S_R$ values were obtained by dividing the plate load by the resilient surface deflection measured on the load plate and are reported in this report in units of kips per inch of deflection. The deflections upon which the stiffness determinations were based are recoverable (or elastic). Any non-recoverable (or permanent) deformations were not used in the determination of $S_R$. Load repetitions were applied to the pavement, and the elastic deflection at the 500th load application was used to determine the stiffness values.

The preconstruction $S_R$ values varied from a low of 91 kips/in. at station 63+00 to a high of 284 kips/in. at 46+00. Data obtained in March were generally in the 100s and in April in the 200s. This is believed to reflect the influence of the thaw recovery period on the pavement's resilient stiffness. The highest pavement temperature associated with these data was 76.1°F; however, most temperature readings varied between 45° and 57°F.

### Frost penetration

The frost penetrations indicated by the thermocouples at stations 43+00 and 63+00 for each of the three years of this test are given in Figures 9 and 10, respectively. Air freezing indices are also shown in these figures. Both the maximum total depth of frost penetration and the maximum frost penetration into the subgrade are listed in Table 2.

**Table 2. Maximum depths of frost penetration.**

| | *1978–1979* | | *1979–1980* | | *1981–1982* | |
|---|---|---|---|---|---|---|
| Freezing index (°F-days) | 1205 | | 975 | | 1284 | |
| Frost penetration (in.) | 56* | 56† | 34* | 36† | 57* | 56† |
| Subgrade frost (in.) | 24* | 8† | 2* | 0† | 25* | 8† |

*Station 43+00
†Station 63+00

The relationship between frost penetration and air freezing index is apparent. During the second year, 1979–1980, which had a lower freezing index than the mean, there was essentially no frost penetration into the underlying subgrade. The other two years had very similar freezing indices and experienced essentially the same total frost penetrations. Although not measured, the maximum subgrade frost penetration at station 54+00 should be approximately midway between the other two stations and is estimated to have been 16, 0, and 17 in. for 1978–1979, 1979–1980 and 1981–1982, respectively.

### Frost heave

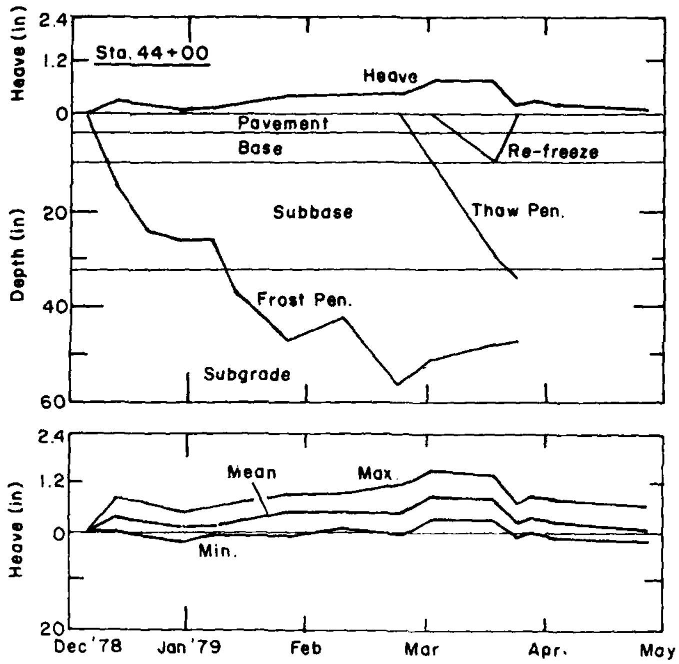
The heaves of the pavement cross sections at stations 44+00, 54+00 and 64+00 are shown in Figures 11, 12 and 13. The elevation data points used to prepare these curves are listed in Tables 3, 4 and 5. Elevations taken prior to the placement of a frost-free bench mark (summer 1979) have been recalculated so that all elevations are relative to the frost-free bench mark shown in Figure 1. Figures 11–13 indicate that the pavements are essentially at the same elevation at the end of the testing program as they were upon completion of the runway reconstruction.

Figures 11–13 also compare the reconstructed pavement elevations prior to the first freeze-thaw season to the maximum heave elevations observed during the test program (March 1982). Except for the mild 1979/1980 winter at station 44+00, heaving of the existing shoulders occurred each year at all three stations. The differential heave between the shoulder and runway is most obvious at stations 54+00 and 64+00 (Fig. 12 and 13). A longitudinal pavement crack, resulting from differential heave between the new runway and existing shoulders, developed during the 1978–1979 winter along the runway/shoulder interface. This interrupted the runway surface drainage path and caused ponding (see Fig. 14).

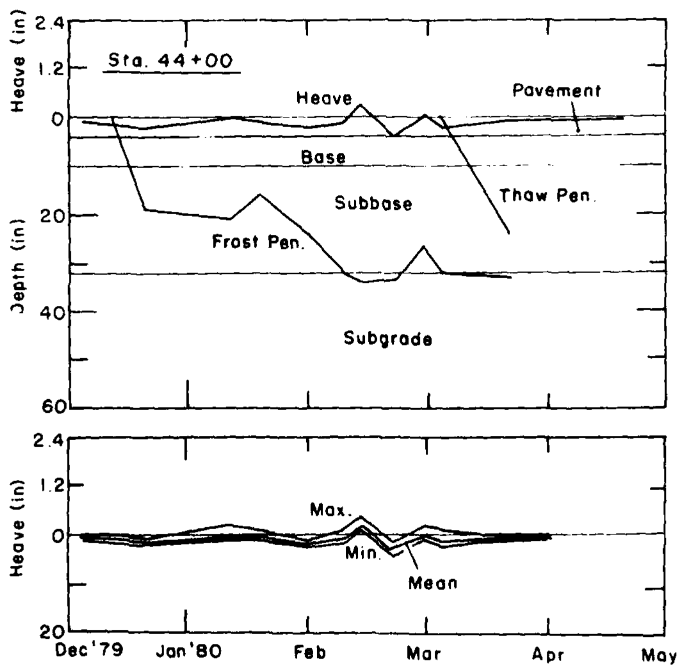
Figures 9 and 10 show heave occurring throughout the course of the winter season at stations 44+00 and 64+00. As would be expected, the greatest heave developed during the winter of highest freezing index (1981–1982) at the thinnest pavement section (station 44+00), where the frost penetrated most deeply into the subgrade. Figure 9 also shows the mean, maximum and minimum heaves measured across station 44+00 at each time the level surveys were run. A maximum heave of 2.9 in. was measured in the shoulder area on 1 March 1982; the mean maximum heave of the runway pavement, excluding the shoulders, was 2.0 in.

The heaving was primarily associated with frost penetration into the subgrade, indicating the high frost susceptibility of that material. During the mild winter of 1979–1980, frost did not penetrate into the subgrade at station 44+00 and essentially no heave was observed. In fact, there appears to have been a slight consolidation produced during freezing of the subbase course that year. The same apparent consolidation is also observed for station 64+00 during the period of subbase freezing in each of the three years of observation (Fig. 10). However, heave, not consolidation, was observed in the shoulders at station 64+00 for each of the three test winters (Fig. 13).

The apparent consolidation, or depression, of the pavement surface during periods of freezing when heave is expected may be due to the use of temporary bench marks which were not frost free. Should this be the case, the reported values of total heave may be underestimated; however, this would not affect differential heaves measured on the same date.

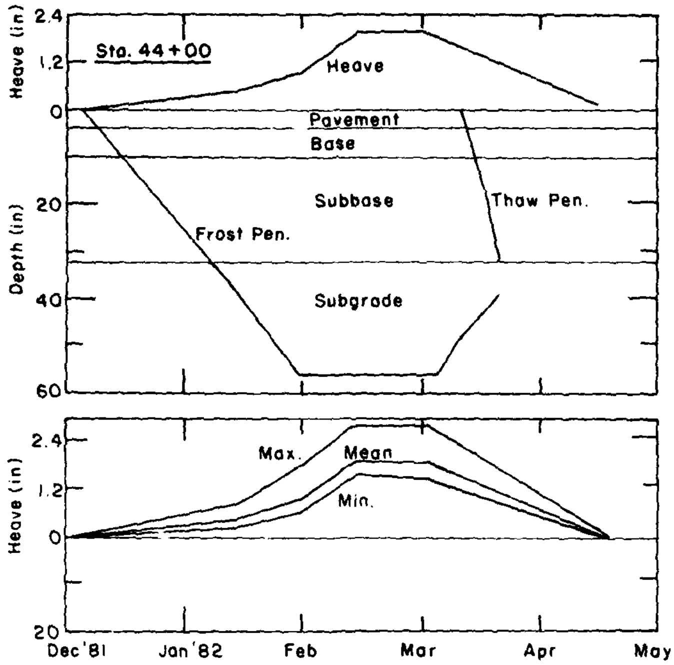



*a. 1978–79 winter season.*



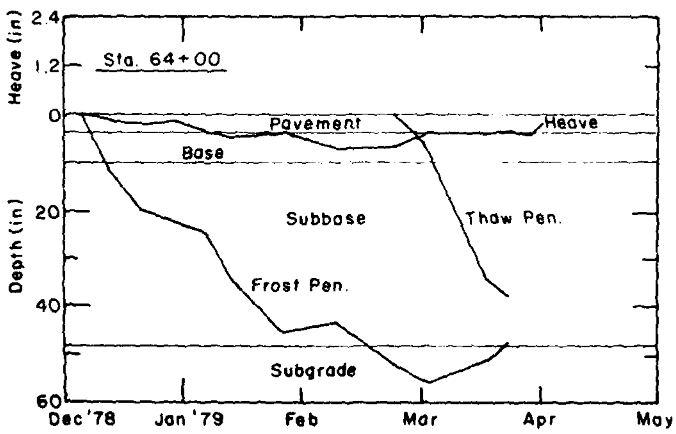

*b. 1979–80 winter season.*

***Figure 9. Frost penetration and heave vs time for three years at station 44+00.***



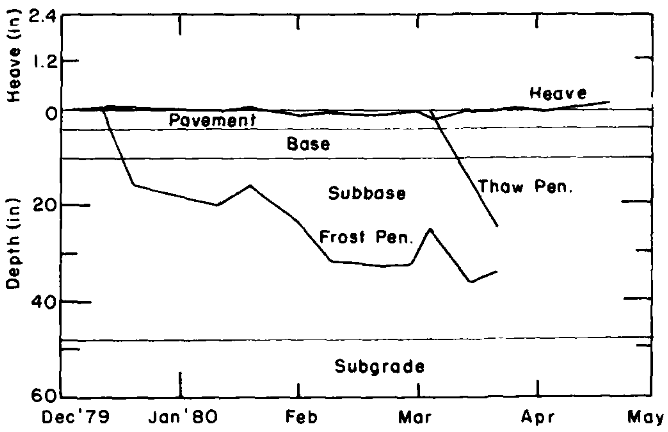

*c. 1981–82 winter season.*

*Figure 9 (cont'd).*

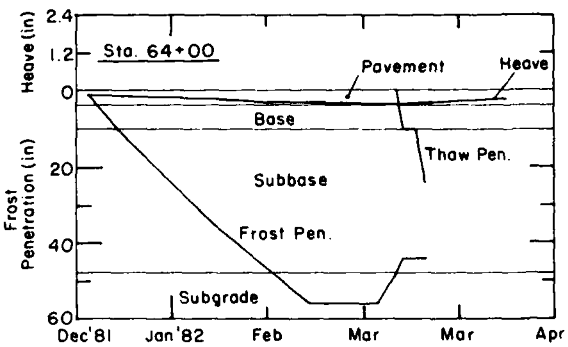



*a. 1978–79 winter season.*

***Figure 10. Frost penetration and heave vs time for three years, station 64+00.***




*b. 1979–80 winter season.*




*c. 1981–82 winter season.*

*Figure 10 (cont'd). Frost penetration and heave vs time for three years, station 64+00.*

**Table 3. Runway elevations (ft)—station 44+00.**

| *Marker* | | *21 Nov 78* | *1 Mar 79* | *9 May 79* | *27 Feb 80* | *18 Apr 80* | *1 Mar 82* | *27 May 82* |
|---|---|---|---|---|---|---|---|---|
| Shoulder | 0 | 110.53 | 110.60 | 110.50 | 110.54 | 110.49 | 110.66 | 110.49 |
| | 5 | 110.57 | 110.64 | 110.54 | 110.57 | 110.53 | 110.75 | 110.54 |
| | 10 | 110.62 | 110.68 | 110.59 | 110.60 | 110.58 | 110.81 | 110.59 |
| | 15 | 110.66 | 110.73 | 110.63 | 110.64 | 110.63 | 110.85 | 110.64 |
| | 20 | 110.71 | 110.78 | 110.70 | 110.70 | 110.68 | 110.89 | 110.68 |
| | 25 | 110.78 | 110.87 | 110.83 | 110.82 | 110.81 | 111.01 | 110.82 |
| Runway | 35 | 110.91 | 111.00 | 110.90 | 110.90 | 110.88 | 111.07 | 110.88 |
| | 45 | 111.05 | 111.10 | 111.03 | 111.04 | 111.03 | 111.19 | 111.04 |
| | 55 | 111.20 | 111.23 | 111.19 | 111.19 | 111.18 | 111.31 | 111.18 |
| | 65 | 111.34 | 111.34 | 111.30 | 111.29 | 111.29 | 111.41 | 111.29 |
| | 75* | 111.42 | 111.45 | 111.41 | 111.41 | 111.41 | 111.54 | 111.41 |
| | 85 | 111.28 | 111.29 | 111.26 | 111.26 | 111.25 | 111.37 | 111.27 |
| | 95 | 111.16 | 111.17 | 111.14 | 111.14 | 111.13 | 111.26 | 111.14 |
| | 105 | 110.99 | 111.64 | 110.98 | 110.99 | 110.97 | 111.15 | 110.99 |
| | 115 | 110.84 | 110.91 | 110.82 | 110.84 | 110.81 | 111.05 | 110.83 |
| Shoulder | 125 | 110.69 | 110.77 | 110.66 | 110.68 | 110.64 | 110.88 | 110.66 |
| | 130 | 110.63 | 110.73 | 110.61 | 110.64 | 110.59 | 110.83 | 110.59 |
| | 135 | 110.58 | 110.68 | 110.55 | 110.57 | 110.54 | 110.78 | 110.55 |
| | 140 | 110.53 | 110.63 | 110.51 | 110.53 | 110.50 | 110.72 | 110.51 |
| | 145 | 110.48 | 110.57 | 110.47 | 110.49 | 110.44 | 110.63 | 110.45 |
| | 150 | 110.44 | 110.53 | 110.49 | 110.51 | 110.47 | 110.65 | 110.49 |

*Centerline

Permanent benchmark at station 54+00.

## Table 4. Runway elevations (ft)—station 54+00.

| Marker | | 21 Nov 78 | 21 Feb 79 | 9 May 79 | 27 Feb 80 | 18 Apr 80 | 1 Mar 82 | 27 May 82 |
|---|---|---|---|---|---|---|---|---|
| Shoulder | 0 | 100.19 | 100.28 | 100.21 | 100.32 | 100.22 | 100.40 | 100.27 |
| | 5 | 100.25 | 100.35 | 100.25 | 100.39 | 100.26 | 100.42 | 100.28 |
| | 10 | 100.32 | 100.39 | 100.32 | 100.44 | 100.32 | 100.48 | 100.34 |
| | 15 | 100.38 | 100.44 | 100.38 | 100.49 | 100.38 | 100.54 | 100.40 |
| | 20 | 100.41 | 100.49 | 100.44 | 100.57 | 100.45 | 100.61 | 100.47 |
| | 25 | 100.50 | 100.55 | 100.50 | 100.60 | 100.56 | 100.65 | 100.53 |
| Runway | 35 | 100.66 | 100.64 | 100.66 | 100.66 | 100.65 | 100.69 | 100.68 |
| | 45 | 100.82 | 100.80 | 100.81 | 100.81 | 100.81 | 100.86 | 100.84 |
| | 55 | 100.95 | 100.94 | 100.95 | 100.94 | 100.95 | 101.00 | 100.96 |
| | 65 | 101.10 | 101.09 | 101.09 | 101.09 | 101.09 | 101.12 | 101.10 |
| | 75* | 101.22 | 101.24 | 101.22 | 101.21 | 101.21 | 101.26 | 101.23 |
| | 85 | 101.09 | 101.09 | 101.10 | 101.09 | 101.09 | 101.12 | 101.11 |
| | 95 | 100.97 | 100.96 | 100.97 | 100.97 | 100.97 | 101.02 | 100.98 |
| | 105 | 100.85 | 100.84 | 100.85 | 100.84 | 100.85 | 100.91 | 100.86 |
| | 115 | 100.73 | 100.73 | 100.73 | 100.73 | 100.73 | 100.80 | 100.75 |
| Shoulder | 125 | 100.60 | 100.63 | 100.61 | 100.70 | 100.62 | 100.73 | 100.63 |
| | 130 | 100.52 | 100.58 | 100.54 | 100.63 | 100.54 | 100.66 | 100.55 |
| | 135 | 100.46 | 100.55 | 100.47 | 100.58 | 100.48 | 100.60 | 100.49 |
| | 140 | 100.40 | 100.51 | 100.40 | 100.52 | 100.40 | 100.55 | 100.41 |
| | 145 | 100.34 | 100.46 | 100.32 | 100.46 | 100.33 | 100.48 | 100.34 |
| | 150 | 100.25 | 100.37 | 100.28 | 100.42 | 100.30 | 100.44 | 100.30 |

*Centerline

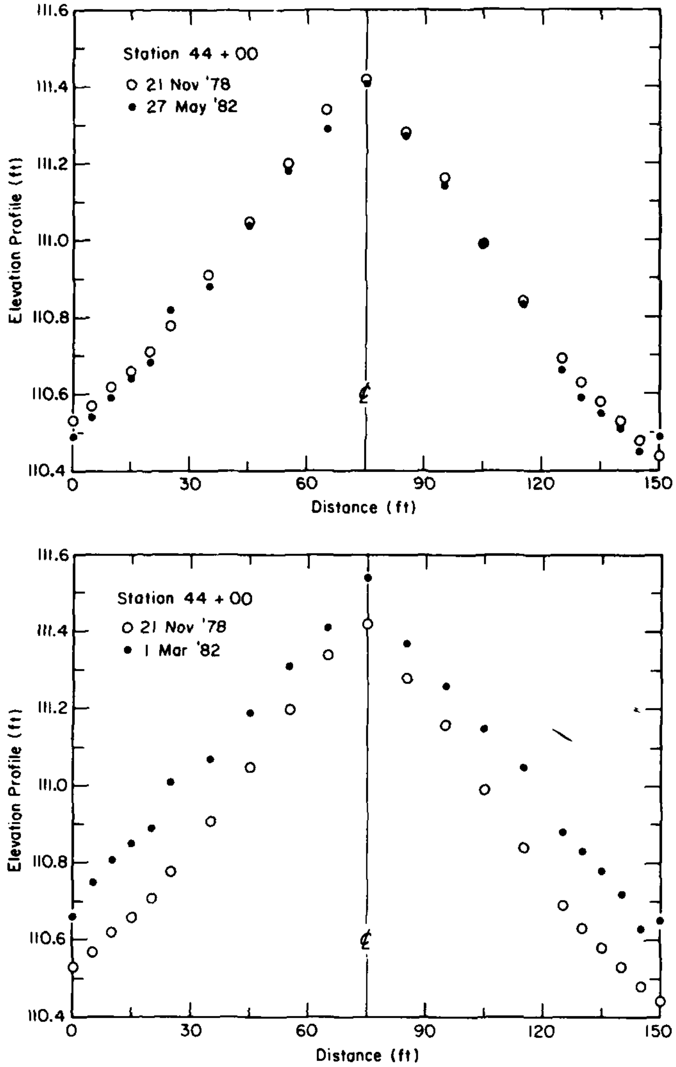
Permanent benchmark at station 54+00.

## Table 5. Runway elevations (ft)—station 64+00.

| Marker | | 21 Nov 78 | 21 Feb 79 | 9 May 79 | 27 Feb 80 | 18 Apr 80 | 1 Mar 82 | 27 May 82 |
|---|---|---|---|---|---|---|---|---|
| Shoulder | 0 | 93.75 | 93.86 | 93.77 | 93.82 | 93.78 | 93.89 | 93.79 |
| | 5 | 93.82 | 93.95 | 93.81 | 93.90 | 93.84 | 93.98 | 93.84 |
| | 10 | 93.89 | 94.04 | 93.89 | 93.98 | 93.91 | 94.06 | 93.91 |
| | 15 | 93.96 | 94.12 | 93.96 | 94.02 | 93.97 | 94.11 | 93.97 |
| | 20 | 94.04 | 94.14 | 94.03 | 94.10 | 94.05 | 94.18 | 94.05 |
| | 25 | 94.13 | 94.25 | 94.13 | 94.21 | 94.14 | 94.26 | 94.12 |
| Runway | 35 | 94.23 | 94.17 | 94.21 | 94.21 | 94.23 | 94.20 | 94.22 |
| | 45 | 94.31 | 94.25 | 94.29 | 94.29 | 94.31 | 94.27 | 94.29 |
| | 55 | 94.41 | 94.35 | 94.39 | 94.39 | 94.41 | 94.37 | 94.39 |
| | 65 | 94.54 | 94.47 | 94.51 | 94.50 | 94.52 | 94.48 | 94.50 |
| | 75* | 94.67 | 94.60 | 94.64 | 94.64 | 94.66 | 94.63 | 94.65 |
| | 85 | 94.54 | 94.47 | 94.52 | 94.51 | 94.54 | 94.50 | 94.52 |
| | 95 | 94.41 | 94.34 | 94.39 | 94.39 | 94.41 | 94.37 | 94.39 |
| | 105 | 94.32 | 94.24 | 94.30 | 94.29 | 94.31 | 94.28 | 94.30 |
| | 115 | 94.24 | 94.17 | 94.23 | 94.22 | 94.24 | 94.21 | 94.22 |
| Shoulder | 125 | 94.19 | 94.28 | 94.20 | 94.2[illegible] | 94.23 | 94.33 | 94.21 |
| | 130 | 94.09 | 94.20 | 94.09 | 94.19 | 94.11 | 94.24 | 94.09 |
| | 135 | 93.99 | 94.11 | 94.00 | 94.08 | 94.01 | 94.15 | 94.00 |
| | 140 | 93.89 | 94.03 | 93.90 | 93.99 | 93.92 | 94.05 | 93.90 |
| | 145 | 93.81 | 93.95 | 93.80 | 93.89 | 93.83 | 93.97 | 93.80 |
| | 150 | 93.72 | 93.84 | 93.73 | 93.79 | 93.75 | 93.88 | 93.75 |

*Centerline

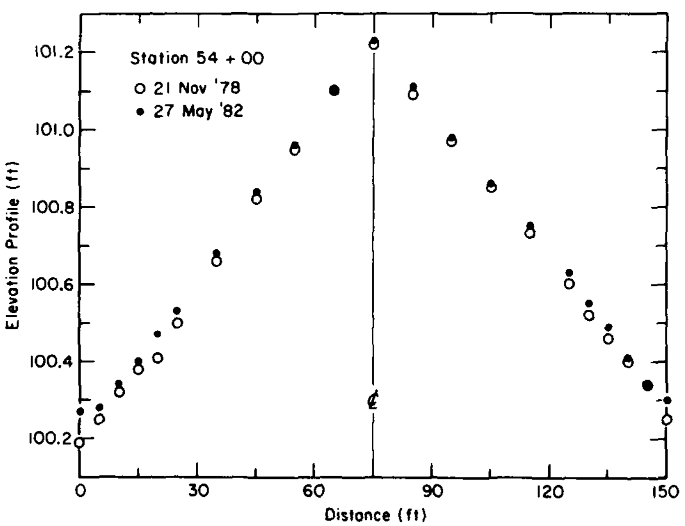
Permanent benchmark at station 54+00.




a. *Initial and final elevations.*

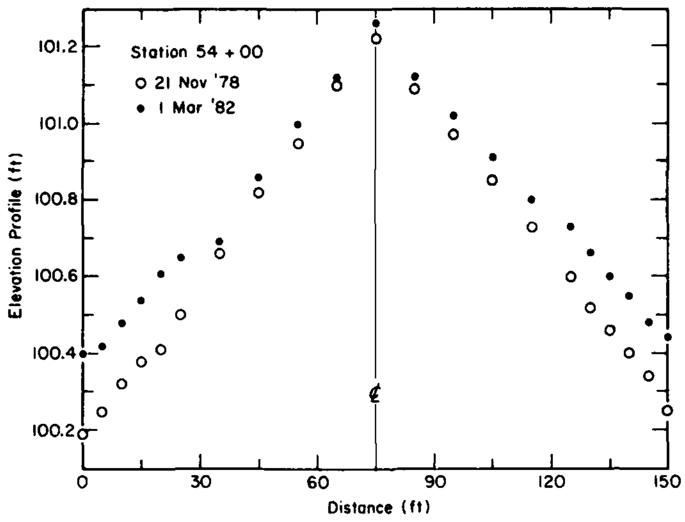
b. *Initial and maximum elevations.*

*Figure 11. Station 44+00, initial, final and maximum pavement elevations.*



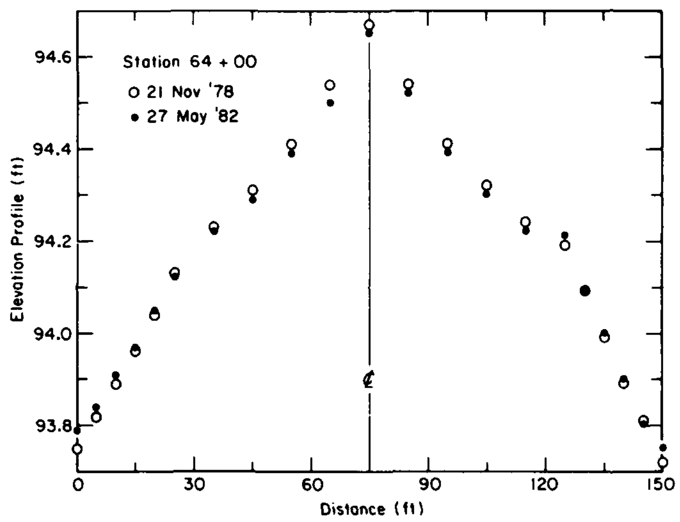

a. *Initial and final elevations.*

*Figure 12. Station 54+00, initial, final and maximum pavement elevations.*

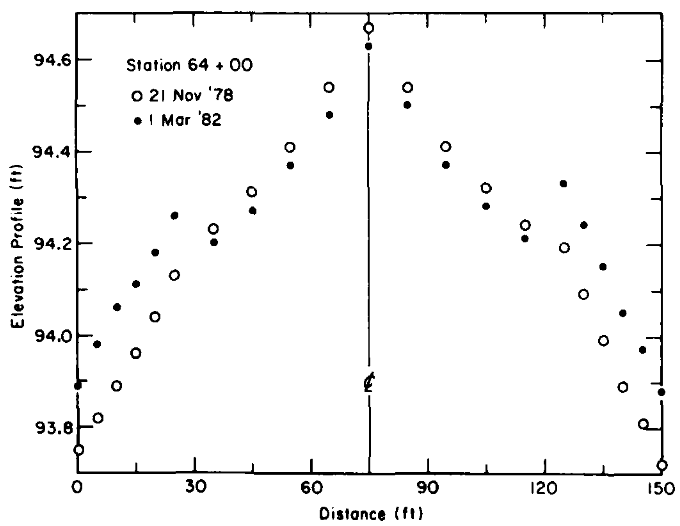



*b. Initial and maximum pavement elevations.*

*Figure 12 (cont'd). Station 54+00, initial, final and maximum pavement elevations.*




*a. Initial and final elevations.*




*b. Initial and maximum elevations.*

*Figure 13. Station 64+00, initial, final and maximum pavement elevations.*

*Figure 14. Longitudinal crack between shoulder and runway.*

**Repetitive plate bearing tests**

Tables 6, 7 and 8 list values of the pavement structure's resilient stiffness measured at stations 44+00, 54+00 and 64+00, respectively. In addition, the tables give the date of observation, pavement temperature, plate load, and the frost and thaw depths at the time of the RPB tests. The stiffness values and the corresponding frost and thaw depths are graphically depicted in Figures 15, 16 and 17 for each station.

Values of $S_R$ in excess of 1500 kips/in. generally represent conditions when the depth of frost is within or below the subbase and thaw has not yet progressed beyond the surface course; winter stiffness values exceeded 3000 kips/in. at all three stations each year, with several observations exceeding 10,000-kips/in. For pavement temperatures of around 16°F and frost penetrations at their maximums of about 4.5 ft, stiffnesses of 30,000-kips/in. were recorded at stations 54+00 and 64+00.

Figure 18 shows the response of the reconstructed pavement's $S_R$ during the spring thaw period, which occurred in March of each year. This figure shows that for each station a minimum pavement stiffness occurs during the spring melting period after which the stiffness values begin to increase. This time is commonly referred to as the thaw recovery period. This minimum $S_R$ can be attributed to the influence of excess water in the pavement structure during a period of relatively rapid thawing. Water released by thawing of the soil in the pavement structure is trapped by the underlying frozen soils, which temporarily create a saturated soil layer within the pavement structure until the water can drain from the section.

Saturation of the soil layer(s) within the pavement structure reduces the effective strength of the soil and thus lowers the load carrying capacity of the pavement section. As the entire section completely thaws and water within the section is drained, the $S_R$ value increases. During the spring thaw period the measured stiffnesses were lowest (400 kips/in. in 1979) for the thinnest (32-in.) pavement section at station 44+00, and highest (610 kips/in. in 1979) for the intermediate (40-in.) section at station 54+00. The thickest (48-in.) pavement section at station 64+00 had a minimum $S_R$ of 589 kips/in. in 1979. The relationship between the $S_R$ values for the three sections remained consistent over the three observational years, with the thinnest section always having the lowest $S_R$ and the intermediate section the highest.

There are several possible explanations for the higher thaw season strength of the 40-in. pavement section compared to the 48-in. section. Lower quality construction of the thicker section in the position of the test point, lesser pavement thickness or an isolated section of weaker subgrade material may all be contributing factors. It would have been preferable to have several replicate RPB test points in each pavement section.

**Table 6. Resilient stiffness measurements, station 44+00, 10 ft left.**

| Date | Pvt temp (°C) | Pvt temp (°F) | $S_R$ (kips/in.) | Load (lb) | Frost pen. (in.) | Thaw pen. (in.) |
|---|---|---|---|---|---|---|
| 3/29/78* | 13.6 | 56.5 | 111 | 8750 | — | — |
| 9/13/78† | 18.3 | 64.9 | 526 | 9000 | 0 | 0 |
| 12/20/78 | −3.2 | 26.2 | 3032 | 9050 | 19 | 0 |
| 12/28/78 | −1.4 | 29.5 | 3186 | 9000 | 22 | 0 |
| 1/5/79 | −5.4 | 22.3 | 5129 | 9000 | 27 | 0 |
| 1/12/79 | −9.2 | 15.4 | 6886 | 9000 | 32 | 0 |
| 1/24/79 | −3.6 | 25.5 | 8240 | 9000 | 46 | 0 |
| 2/7/79 | −8.8 | 16.2 | 4932 | 9000 | 42 | 0 |
| 2/21/79 | −3.1 | 26.4 | 6368 | 8750 | 53 | 0 |
| 3/1/79 | +1.5 | 34.7 | 781 | 9000 | 52 | 5.5 |
| 3/16/79 | 6.4 | 43.5 | 432 | 9000 | 10/48** | 29 |
| 3/22/79 | 13.7 | 56.7 | 400 | 9000 | 46.5 | 34 |
| 3/26/79 | 9.3 | 48.7 | 480 | 9100 | 45 | 38 |
| 3/30/79 | 7.2 | 45.0 | 542 | 9000 | 0 | 0 |
| 4/25/79 | 21.1 | 70.0 | 372 | 9000 | — | — |
| 5/9/79 | 21.7 | 71.1 | 408 | 9000 | — | — |
| 7/23/79 | 43.0 | 109.4 | 322 | 9300 | — | — |
| 8/28/79 | 38.4 | 101.1 | 428 | 9750 | — | — |
| 10/5/79 | 19.0 | 66.2 | 536 | 9700 | — | — |
| 10/31/79 | 10.3 | 50.5 | 557 | 9700 | — | — |
| 11/14/79 | 7.7 | 45.9 | 723 | 9900 | — | — |
| 12/3/79 | 4.1 | 39.4 | 713 | 9200 | — | — |
| 12/11/79 | 4.5 | 40.1 | 756 | 9150 | 0 | 0 |
| 12/19/79 | −5.4 | 22.3 | 3814 | 9000 | 13 | 0 |
| 1/9/80 | −1.6 | 29.1 | 3813 | 9000 | 20 | 0 |
| 1/17/80 | +1.4 | 34.5 | 1531 | 9125 | 17 | 4†† |
| 1/29/80 | +1.6 | 34.9 | 3814 | 9000 | 21 | 4†† |
| 2/7/80 | −1.6 | 29.1 | 8505 | 9100 | 32 | 0 |
| 2/11/80 | −4.4 | 24.1 | 5732 | 9000 | 33 | 0 |
| 2/19/80 | 1.1*** | 34.0 | 5921 | 9000 | 34 | 0 |
| 2/21/80 | −5.6 | 21.9 | 11270 | 8900 | 34 | 0 |
| 3/3/80 | −1.8 | 28.8 | 17310 | 9000 | 34 | 0 |
| 3/20/80 | 12.8 | 55.0 | 521 | 9500 | 0 | 0 |
| 3/24/80 | 11.4 | 52.5 | 626.8 | 9500 | 0 | 0 |
| 3/26/80 | 7.0 | 44.6 | 666 | 9000 | — | — |
| 3/28/80 | 11.3 | 52.3 | 568 | 9150 | — | — |
| 3/31/80 | 10.1 | 50.2 | 576.9 | 9000 | — | — |
| 4/2/80 | 7.8 | 46.0 | 641 | 9000 | — | — |
| 4/18/80 | 11.8 | 53.2 | 596 | 9050 | — | — |
| 5/29/80 | 30.1 | 86.2 | 454 | 9000 | 0 | 0 |
| 3/1/82 | −1.2 | 29.8 | 22500 | 9000 | 55 | 0 |
| 3/10/82 | 4.5 | 40.1 | 3000 | 8750 | 48 | 13 |
| 3/16/82 | 11.0 | 51.8 | 652 | 8250 | 42 | 22 |
| 3/19/82 | 12.9 | 55.2 | 449 | 8250 | 42 | 31 |
| 3/24/82 | 14.1 | 57.4 | 412 | 8250 | 0 | 0 |
| 3/30/82 | 10.0 | 50.0 | 493 | 8500 | — | — |
| 11/28/82 | −7.7 | 18.1 | 3000 | 8900 | 55 | 0 |

* 44+00 15 ft right (preconstruction)
† 44+00 10 ft right
** 0 in. to 10 in. frozen
10 in. to 29 in. thawed
29 in. to 48 in. frozen
†† Pvt thawed (above freezing)
*** Top 1/3 of pvt above freezing

**Table 7. Resilient stiffness measurements, station 54+00, 10 ft left.**

| *Date* | *Pvt temp (°C)* | *(°F)* | $S_R$ *(kips/in.)* | *Load (lb)* | *Frost pen. (in.)* | *Thaw pen. (in.)* |
|---|---|---|---|---|---|---|
| 3/31/78* | 8.2 | (46.8) | 108.9 | 8800 | — | — |
| 9/18/78 | 17.7 | (63.9) | 532 | 9000 | 0 | 0 |
| 12/20/78 | — | | 2762 | 9150 | 19.5 | 0 |
| 12/28/78 | — | | 3290 | 9000 | 21.0 | 0 |
| 1/5/79 | — | | 3008 | 8900 | 29.5 | 0 |
| 1/12/79 | — | | 5208 | 9000 | 32.8 | 0 |
| 1/24/79 | — | | 10933 | 9000 | 40.0 | 0 |
| 2/7/79 | — | | 4776 | 9200 | 40.2 | 0 |
| 2/21/79 | — | | 6870 | 8850 | 49.0 | 0 |
| 3/1/79 | — | | 928 | 9200 | 51.0 | 10 |
| 3/16/79 | 6.4 | (43.5) | 662 | 9050 | 10/48† | 29 |
| 3/22/79 | 13.7 | (56.7) | 610 | 9000 | 46.5 | 34 |
| 3/24/79 | 11.4 | (52.5) | 621 | 9500 | 0 | 0 |
| 3/26/79 | 9.3 | (48.7) | 708 | 9000 | 45.0 | 38 |
| 3/30/79 | — | | 705 | 9000 | 0 | 0 |
| 4/25/79 | — | | 561 | 9000 | — | — |
| 7/23/79 | 43.0 | (109.4) | 420 | 9200 | — | — |
| 10/9/79 | 6.7 | (44.1) | 855.3 | 9750 | — | — |
| 10/31/79 | 8.2 | (46.8) | 975 | 9850 | — | — |
| 11/14/79 | 7.8 | (46.0) | 970 | 10000 | — | — |
| 12/3/79 | 2.8 | (37.0) | 981 | 9600 | — | — |
| 12/11/79 | 2.2 | (36.0) | 978 | 9000 | 0 | 0 |
| 12/19/79 | –8.5 | (16.7) | 4053 | 9200 | 13 | 0 |
| 1/9/80 | –3.5 | 25.7 | 2944 | 9100 | 20 | 0 |
| 1/17/80 | –0.2 | 31.6 | 2427 | 9150 | 21 | 0 |
| 1/29/80 | –2.6 | 27.3 | 6667 | 9000 | 25 | 0 |
| 2/7/80 | –9.3 | 15.3 | 5637 | 8850 | 33 | 0 |
| 2/11/80 | –5.1 | 22.8 | 6130 | 8950 | 34 | 0 |
| 2/19/80 | –1.1 | 30.0 | 4516 | 8900 | 34 | 0 |
| 2/27/80 | –8.0 | 17.6 | 10333 | 9350 | 34 | 0 |
| 3/3/80 | –5.6 | 21.9 | 16510 | 8750 | 34 | 0 |
| 3/13/80 | — | | 1127 | 9350 | 8/37** | 15 |
| 3/20/80 | 12.7 | 54.9 | 866 | 9700 | 0 | 0 |
| 3/24/80 | 11.4 | 52.5 | 927 | 8900 | — | — |
| 3/26/80 | 6.0 | 42.8 | 932 | 8950 | — | — |
| 3/28/80 | 8.5 | 47.3 | 883 | 9100 | — | — |
| 3/31/80 | 7.7 | 45.9 | 927 | 9000 | — | — |
| 4/2/80 | 7.4 | 45.3 | 957 | 9000 | — | — |
| 4/18/80 | 8.0 | 46.4 | 973 | 9050 | — | — |
| 5/29/80 | 24.0 | 75.2 | 633 | 9000 | 0 | 0 |
| 1/28/82 | –9 | 15.8 | 30000 | 8900 | 55 | 0 |
| 3/1/82 | –2.5 | 27.5 | 22000 | 9000 | 54 | 0 |
| 3/10/82 | 2.5 | 36.5 | 3225 | 8750 | 46 | 7 |
| 3/12/82 | 3.3 | 37.9 | 1190 | 8850 | 44 | 10 |
| 3/16/82 | 9.5 | 49.1 | 870 | 8750 | 43 | 16 |
| 3/19/82 | 10.8 | 51.4 | 725 | 8500 | 41 | 29 |
| 3/24/82 | 13.5 | 56.3 | 715 | 8500 | 0 | 0 |
| 3/30/82 | 8.5 | 47.3 | 763 | 8250 | — | — |

* 51+90 10 ft right

† 0 in. to 10 in. frozen
10 in. to 29 in. thawed
29 in. to 48 in. frozen

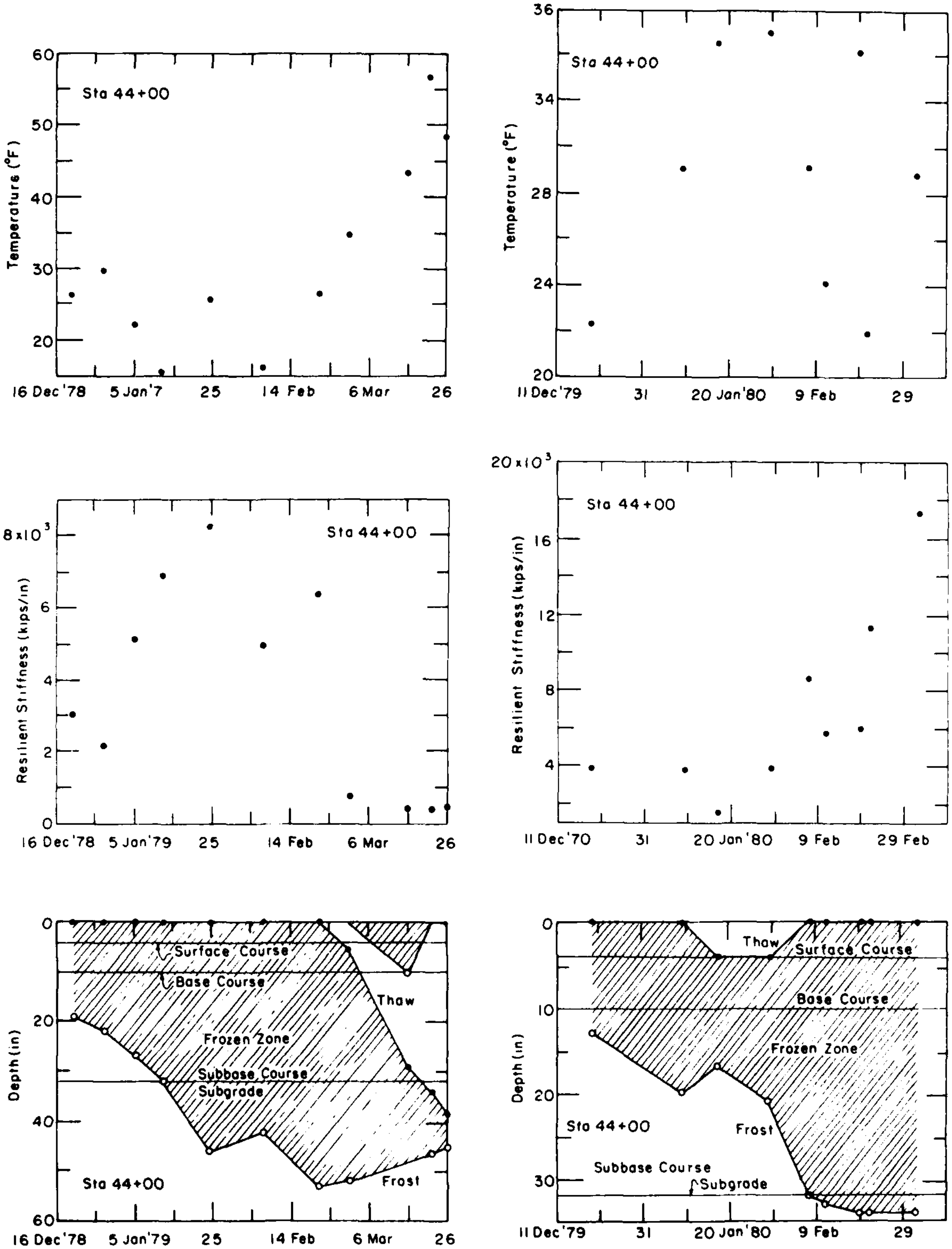
** 0 in. to 8 in. frozen
8 in. to 15 in. thawed
15 in. to 37 in. frozen

**Table 8. Resilient stiffness measurements, station 64+00, 10 ft left.**

| *Date* | *Pvt temp* (°C) | (°F) | $S_R$ *(kips/in.)* | *Load (lb)* | *Frost pen. (in.)* | *Thaw pen. (in.)* |
|---|---|---|---|---|---|---|
| 3/30/78* | 15.6 | 60.1 | 136 | 8750 | — | — |
| 9/18/78† | 26.1 | 79.0 | 405 | 9000 | 0 | 0 |
| 12/20/78 | –1.9 | 28.6 | 2823 | 9050 | 19 | 0 |
| 12/28/78 | –8.1 | 17.4 | 3146 | 9000 | 22 | 0 |
| 1/5/79 | –7.9 | 17.8 | 4528 | 8750 | 25 | 0 |
| 1/12/79 | –11.0 | 12.2 | 4625 | 9000 | 33 | 0 |
| 1/24/79 | –4.6 | 23.7 | 9244 | 9000 | 45 | 0 |
| 2/7/79 | –10.6 | 12.9 | 7262 | 9100 | 44 | 0 |
| 2/21/79 | –5.6 | 21.9 | 4618 | 8950 | 53 | 0 |
| 3/1/79 | 1.4 | 34.5 | 799 | 9000 | 55 | 10 |
| 3/16/79 | 6.6 | 43.9 | 636 | 9100 | 11/51** | 34 |
| 3/22/79 | 9.5 | 49.1 | 589 | 9050 | 47 | 38 |
| 3/26/79 | 8.8 | 47.8 | 636 | 9000 | 0 | 0 |
| 3/30/79 | 7.1 | 44.8 | 625 | 9000 | — | — |
| 4/25/79 | 17.2 | 63.0 | 532 | 9000 | — | — |
| 5/9/79 | 23.2 | 73.8 | 455 | 9000 | — | — |
| 7/23/79 | 43.0 | 109.4 | 399 | 9300 | — | — |
| 8/28/79 | 35.2 | 95.4 | 542 | 9500 | — | — |
| 10/9/79 | 6.7 | 44.1 | 754 | 9750 | — | — |
| 10/31/79 | 6.0 | 42.8 | 734 | 9900 | — | — |
| 11/14/79 | 7.8 | 46.0 | 726 | 9800 | — | — |
| 12/3/79 | 1.5 | 34.7 | 900 | 9900 | — | — |
| 12/11/79 | 2.2 | 36.0 | 763 | 9000 | 0 | 0 |
| 12/19/79 | –8.5 | 16.7 | 3488 | 9000 | 13 | 0 |
| 1/9/80 | –3.5 | 25.7 | 2966 | 9000 | 19 | 0 |
| 1/17/80 | –.2 | 31.6 | 2381 | 9125 | 21 | 0 |
| 1/29/80 | –2.6 | 27.3 | 6741 | 9100 | 25 | 0 |
| 2/7/80 | –4.3 | 24.3 | 5085 | 9000 | 34 | 0 |
| 2/11/80 | –5.1 | 22.8 | 11270 | 8900 | 34 | 0 |
| 2/19/80 | –1.1 | 30.0 | 10340 | 9000 | 35 | 0 |
| 2/27/80 | –8.0 | 17.6 | 8091 | 8900 | 35 | 0 |
| 3/3/80 | –3.5 | 25.7 | 15000 | 8850 | 35 | 0 |
| 3/13/80 | — | | 1533 | 9350 | 8/37†† | 15 |
| 3/20/80 | 6.7 | 44.1 | 789 | 9550 | 0 | 0 |
| 3/24/80 | 6.6 | 43.9 | 694 | 9300 | — | — |
| 3/26/80 | 6.1 | 43.0 | 817 | 9150 | — | — |
| 3/28/80 | 8.5 | 47.3 | 705 | 9100 | — | — |
| 3/31/80 | 7.7 | 45.9 | 727 | 9000 | — | — |
| 4/2/80 | 7.4 | 45.3 | 741 | 9050 | — | — |
| 4/14/80 | — | 752 | 9100 | — | — | |
| 4/18/80 | 8.0 | 46.4 | 784 | 9200 | — | — |
| 5/29/80 | 29.9 | 85.8 | 488 | 8800 | 0 | 0 |
| 1/28/82 | –9.4 | 15.1 | 30000 | 8900 | 55 | 0 |
| 3/1/82 | –2.5 | 27.5 | 15000 | 9000 | 48 | 0 |
| 3/10/82 | 1.8 | 35.2 | 3913 | 9000 | [illegible]4 | 4 |
| 3/12/82 | 3.3 | 37.9 | 1133 | 8500 | 44 | 10 |
| 3/16/82 | 8.1 | 46.6 | 702 | 8000 | 44 | 10 |
| 3/19/82 | 8.8 | 47.8 | 652 | 8500 | 40 | 26 |
| 3/24/82 | 13 | 55.4 | 600 | 8750 | 0 | 0 |
| 3/30/82 | 7.5 | 45.5 | 667 | 8750 | — | — |

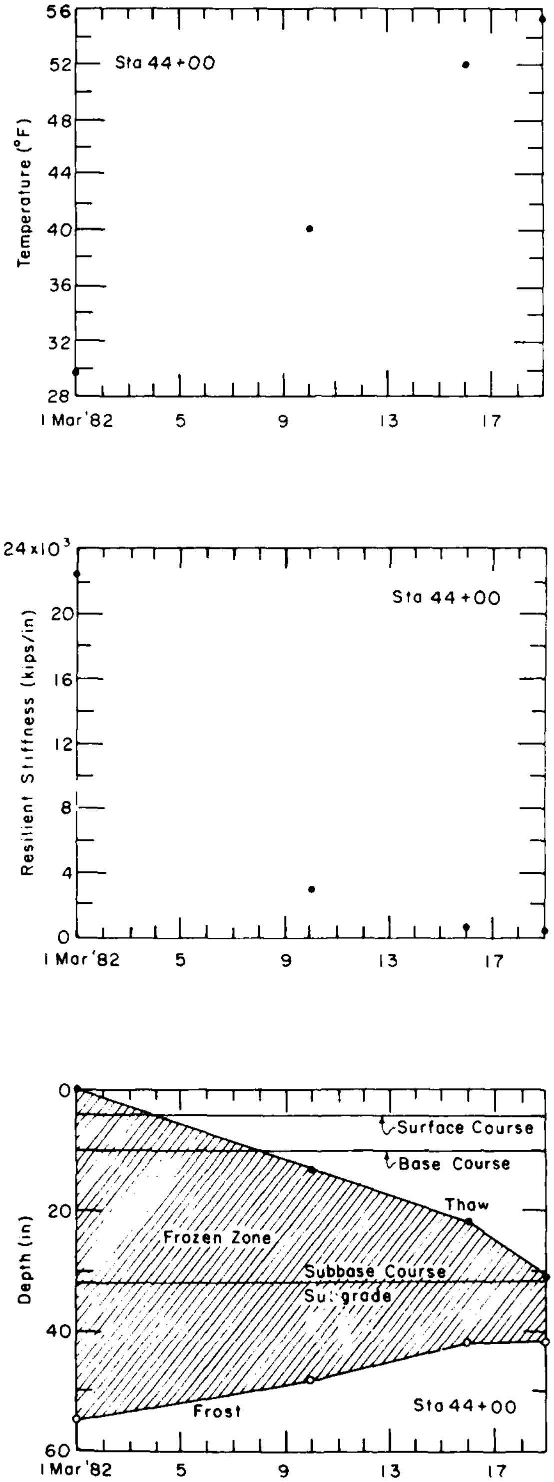
* 63+00 15 ft left
† 64+50 10 ft left
** 0 in. to 11 in. frozen
11 in. to 34 in. thawed
34 in. to 51 in. frozen

†† 0 in. to 8 in. frozen
8 in. to 15 in. thawed
15 in. to 37 in. frozen



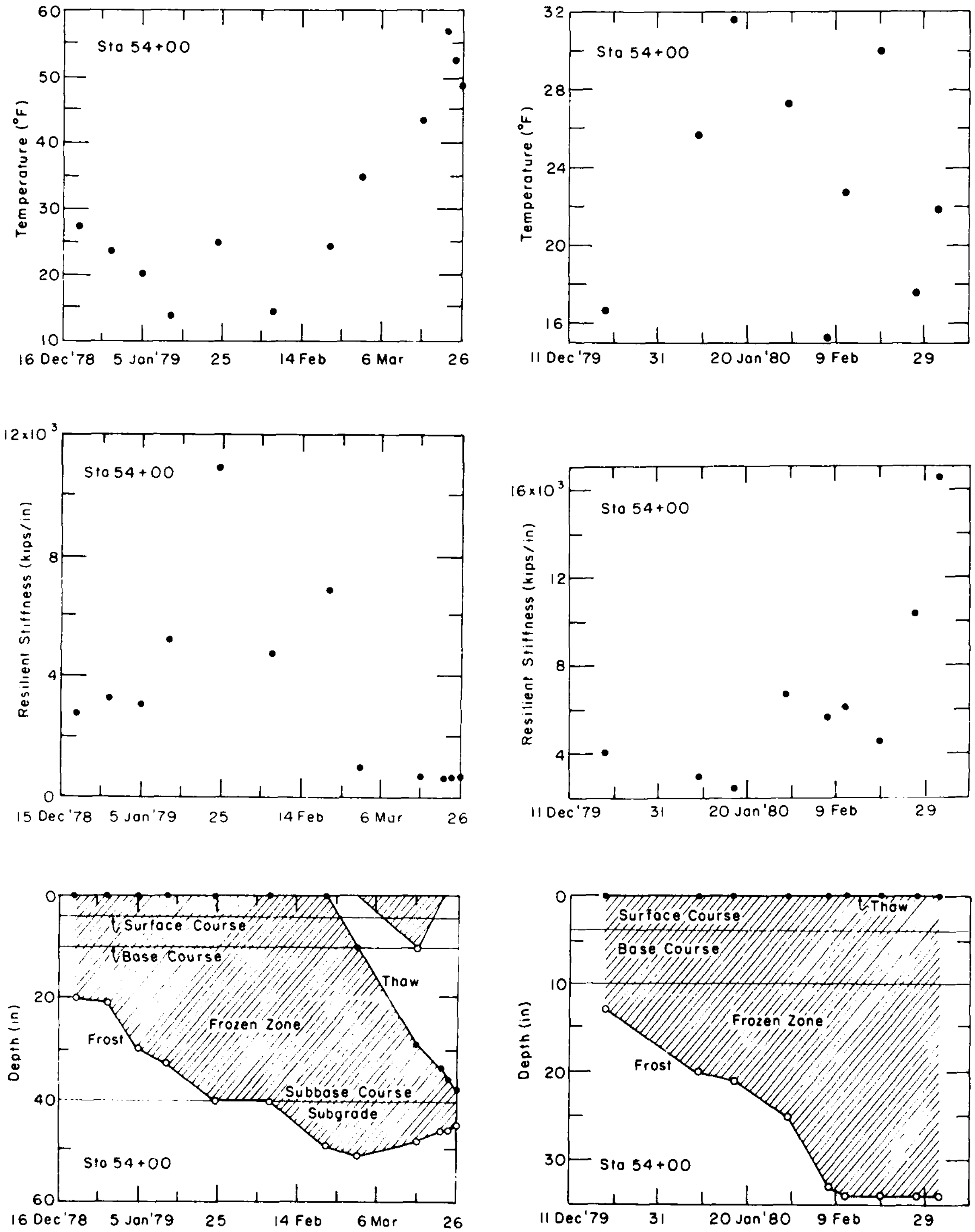

*a. 1978–79 winter season.* *b. 1979–80 winter season.*

*Figure 15. Variation of resilient stiffness with frozen zone change, station 44+00.*




*c. 1981–82 winter season.*

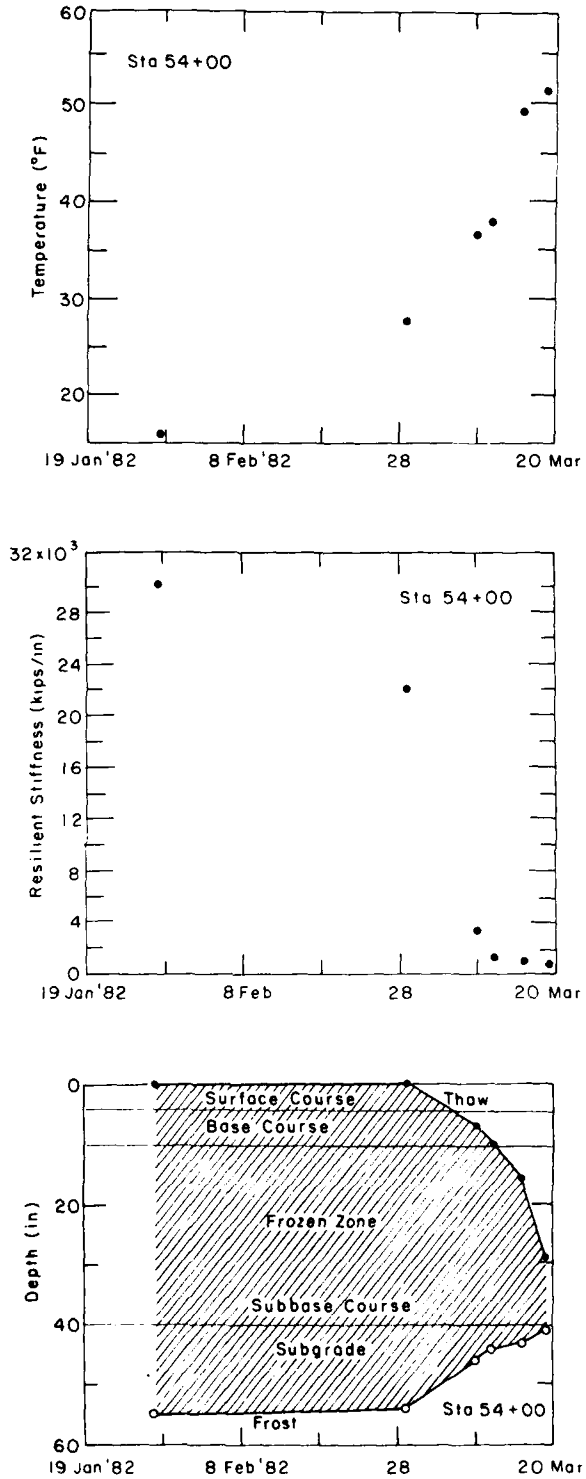
***Figure 15 (cont'd). Variation of resilient stiffness with frozen zone change, station 44+00.***




*a. 1978–79 winter season.*

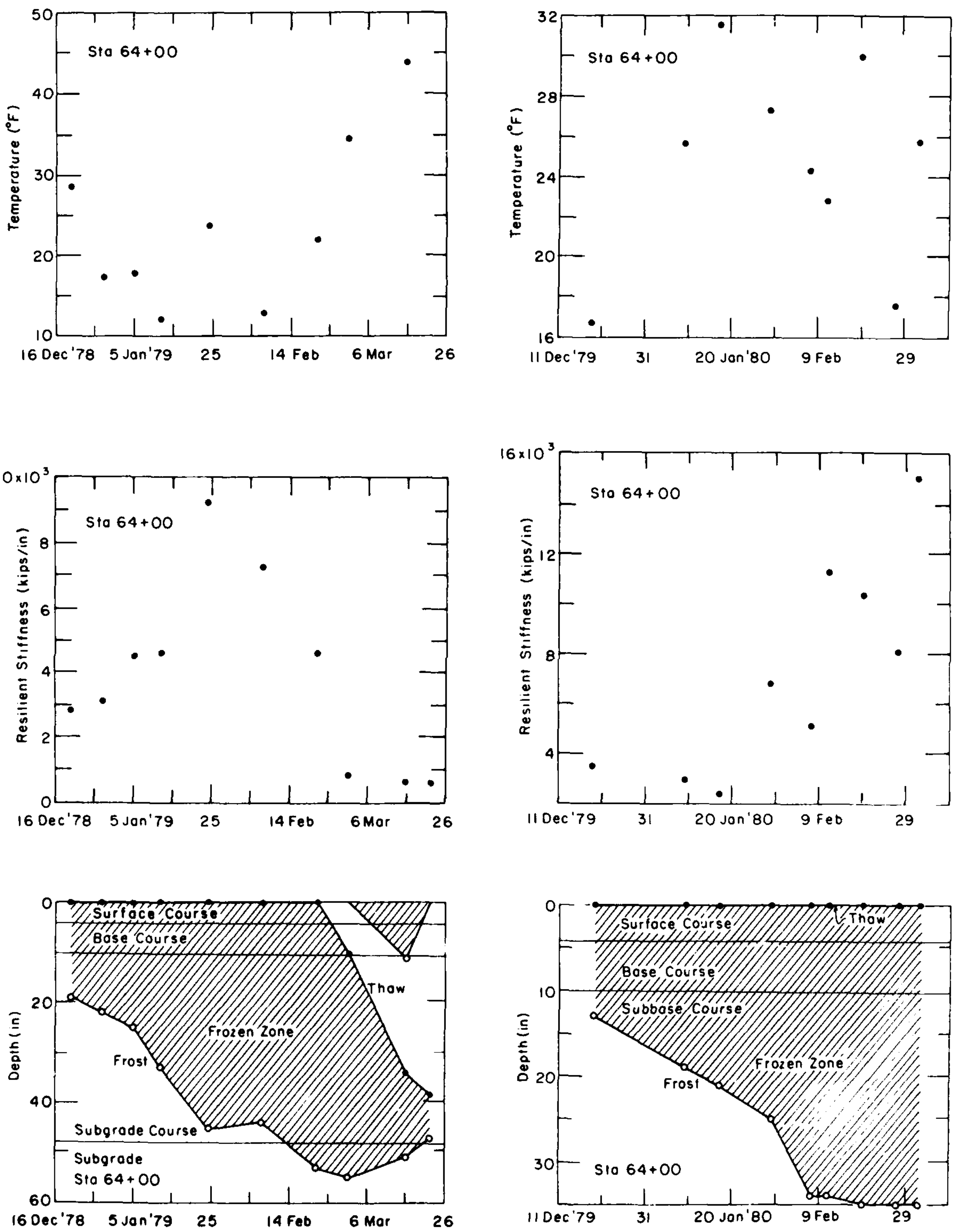
*b. 1979–80 winter season.*

*Figure 16. Variation of resilient stiffness with frozen zone change, station 54+00.*




*c. 1981–82 winter season.*

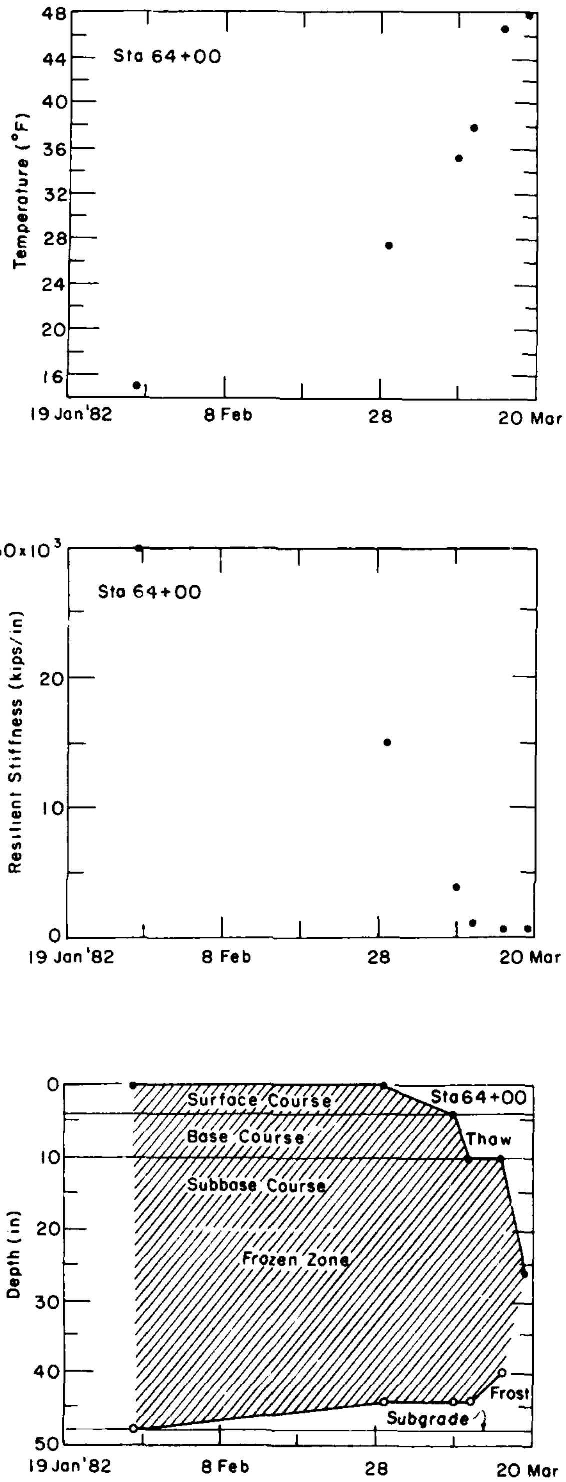
*Figure 16 (cont'd). Variation of resilient stiffness with frozen zonc change, station 54+00.*




*a. 1978–79 winter season.*

*b. 1979–80 winter season.*

***Figure 17. Variation of resilient stiffness with frozen zone change, station 64+00.***




*c. 1981–82 winter season.*

***Figure 17 (cont'd). Variation of resilient stiffness with frozen zone change, station 64+00.***

Overall, March values of $S_R$ were slightly higher for the mild winter of 1979–1980 when the frost penetration was least. This could be attributable to the lack of subgrade frost penetration at stations 54+00 and 64+00 and only a few inches of frost penetration into the thinnest section at station 44+00.

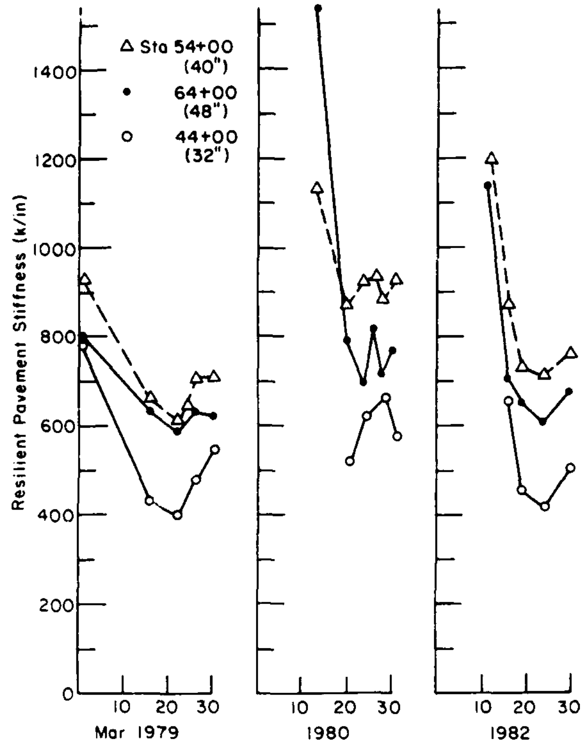



*Figure 18. Resilient stiffness vs time (March), three years.*

The effect of temperature on pavement stiffness during other than the freeze-thaw season is shown in Figure 19. The minimum stiffness values were obtained in July when the pavement temperature was 109.4°F. At these higher temperatures the asphalt concrete itself is less stiff and more likely to deform under loading. Temperature effects on the stiffness of soil layers in the pavement structure are considered negligible. The effect of pavement temperature on asphalt stiffness has been reported by others (Bush 1987, Schmidt 1973).

Although Figure 19 provides some insight as to the effect of temperature on the asphalt pavement's stiffness, it does not include any normalization that would eliminate the contributing effect of subgrade moisture content. Previous work conducted by CRREL has shown the significant influence of moisture content on the strength of silty soils similar to those found at the Lebanon Regional Airport (Johnson et al. 1978). Figure 20 shows the monthly precipitation during the test period. Comparisons of the stiffness values for periods of different levels of precipitation indicate no correlation between pavement stiffness and precipitation. The lack of measured water content test data makes it difficult to quantify the effects of water within the pavement layers on the strength of the pavement structure.

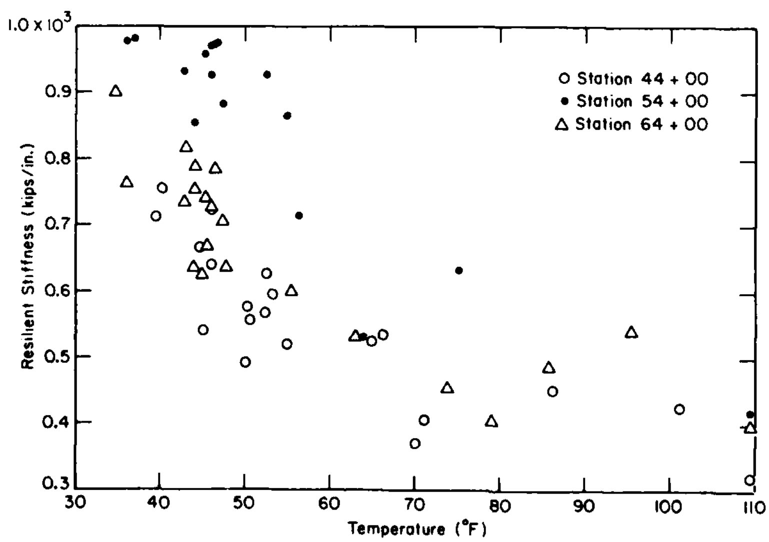



*Figure 19. Resilient stiffness vs pavement temperature.*

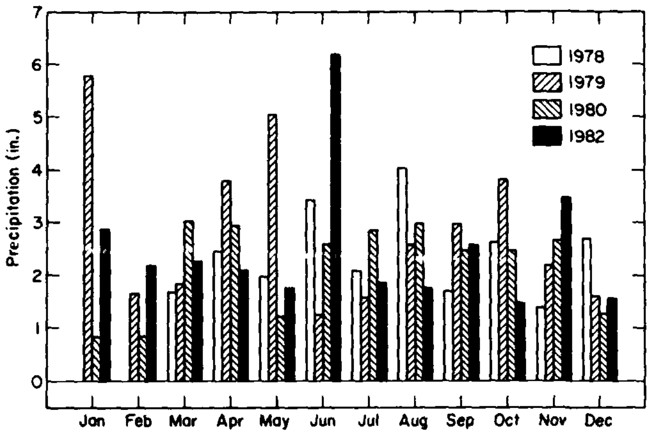



*Figure 20. Monthly precipitation.*

## CONCLUSIONS

The 48-in. pavement section at station 64+00 provided the highest level of frost protection. Even so, frost penetration of up to 8-in. into the subgrade was recorded during the testing period when the freezing index was still 600°F-days or about one-third less than the design freezing index of 1820°F-days.

The frost heaves observed will not cause concern for the aircraft using the Lebanon Regional Airport from a roughness standpoint, but ponding of water on the runway could cause areas of ice which may pose a safety hazard.

With reconstruction of the pavement section at the Lebanon Regional Airport, the load-carrying capabilities of the runway have been increased substantially. Even during the critical spring thaw period the new pavement has a resilient stiffness on the order of two to three times that of the pavement before reconstruction.

Resilient stiffness of the pavement sections is affected by several factors. When frost penetrates through the entire pavement layer, resilient stiffness can be as high as 30,000-kips/in. Thaw weakening of the pavement occurs when the partially thawed zone in the pavement structure becomes saturated. Increased water content of the underlaying soil layers has been shown to decrease the strength of pavement structures (Ridgeway 1982). Increase of the asphalt pavement surface course temperature during the summer caused a decrease in the resilient stiffness as the bituminous concrete softened and became less rigid.

Results of this study provide baseline data that should prove useful in any future pavement evaluation programs at the Lebanon Regional Airport.

## RECOMMENDATIONS

In order to avoid the observed differential heave that occurred between the shoulders and interior portion of the runway, it is recommended that future reconstruction minimize any such abrupt discontinuity in pavement structure by creating a smooth transition within the shoulder sections. The transition should not be sloped more steeply than four horizontal/one vertical and the entire transition should take place in the shoulders.

During the observational program covered by this report, the air freezing index never approached the design level of 1820°F-days. It is suggested that repeated plate bearing load tests be conducted weekly during the thaw weakening period (March) during a year in which air freezing index is in excess of 1700°F-days. This would require tabulating the freezing index over the course of each winter to see if design index conditions were being approached. Such a year would produce deeper frost penetrations in the subgrade, particularly at station 44+00.

Testing of the resilient response at more than a single point within a given test section would decrease the influence of any isolated anomalies that might be present within the pavement structure and also provide a more representative structural response value for the pavement section.

Further investigation of the decrease in stiffness associated with an increase in temperature of the bituminous pavement surface course might also be considered. Repeated plate bearing load tests should be extended throughout the summer and at more frequent intervals to establish any correlation with reported precipitation, and to determine possible effects of increased moisture in the pavement system, and the time intervals associated with the detrimental effects of surface water infiltration into the pavement.

## SUMMARY

The three winters occurring during these tests covered a range of freezing indices from near a mean condition (975°F-days) to somewhat colder than a mean (1205°F-days and 1284°F-days). The highest freezing index of 1284°F-days was well below the design freezing index of 1820°F-days. The pavement stations monitored covered a range of combined subbase, base, and pavement thicknesses of 32-, 40- and 48-in.

The total measured depths of frost penetration ranged from approximately 3-ft during the mildest winter to 4.6-ft during the two colder winters. During the mild winter (1979–1980) there was essentially no frost penetration into the subgrade material. During the colder winters, maximum subgrade frost penetrations of 25, 17 and 8 in. occurred at stations 44+00, 54+00 and 64+00, respectively.

The greatest amount of heave developed in the shoulder areas that were not reconstructed during the 1978 runway rebuilding program. A maximum measured heave of 2.9-in. was observed in the shoulder at station 44+00 (32-in. pavement) during the winter of highest freezing index, 1981–1982. The largest differential heaves were noted between the shoulder and reconstructed runway sections where heave was so severe that longitudinal cracks developed that ran the length of the runway. The cracks developed during the first winter after reconstruction and created ponding of water on the runway. During the coldest winter at station 44+00 a mean maximum heave of 2.0 in. was observed over the runway cross section, excluding the shoulders. No heave in the reconstructed runway section was observed during the mild winter of 1979–1980.

Although some slight heave occurred as frost was penetrating the base and subbase, the bulk of the heave was associated with frost penetrating into the subgrade, thereby demonstrating the frost susceptibility of that material. During freezing of the subbase at station 64+00, a slight "apparent" consolidation was measured. "Apparent" consolidation was also observed at station 44+00 during the mild winter, 1979-1980. This observation was probably the result of frost heave of the bench marks. The pavement sections appeared to recover from heave displacements experienced during the winter seasons; elevations recorded in November 1978 and May 1982 are generally within a few tenths of an inch of each other.

The observed frost heaves generally provided no hazard for the aircraft that routinely use the Lebanon Regional Airport facilities. These aircraft are small, and are designed for use on gravel surfaces.

Since reconstruction, the measured wintertime resilient stiffness values routinely exceeded 3000-kips/in., some being in excess of 10,000-kips/in. A value of 30,000-kips/in. was measured for a pavement temperature around 16°F and frost penetration of 4.5 ft.

Although the resilient stiffness of the reconstructed pavement structure decreased substantially during spring thaw, it remained greater than the $S_R$ values measured prior to reconstruction. The lowest springtime post-construction $S_R$ measurement was 400-kips/in. compared to values in the 100s prior to rebuilding. The spring values of $S_R$ were highest for the 40-in. section and lowest for the 32-in. pavement section. At all three stations during the mild winter of 1979–80, the measured spring $S_R$ values were generally higher than those measured during the other two colder winters.

A comparison of temperature variation and stiffness measurements made throughout the spring, summer and fall of the test years indicated the lowest stiffness occurred at a pavement temperature of 109°F (43°C). The measurements also indicated that during the frost melting period, the stiffness values dropped to a minimum level, probably influenced by saturation of the thawed pavement layers. The pavement stiffness then recovered to higher values during thaw recovery and developed a downward trend in the summer, which is attributed to the influence of higher pavement temperatures on the stiffness of the bituminous surface course.

As a general rule, the pavement strength may be represented by pavement stiffness (or pavement layer modulus). Lower stiffness results in a reduction in the number of design wheel loads a pavement structure can withstand before repair or rehabilitation is necessary. Applications of loadings during times of low pavement strength will contribute the most to accelerated pavement deterioration.

Results of this study provide baseline data that should prove useful in any future pavement evaluation programs at the Lebanon Regional Airport.

## LITERATURE CITED

**Berg, R. L. and T. C. Johnson** (1983) Revised procedure for pavement design under seasonal frost conditions. USA Cold Regions Research and Engineering Laboratory, Special Report 83-27.

**Bush, A. J.** (1987) Development of a pavement evaluation method for low-volume airfield pavements. Ph.D thesis (unpublished), University of Illinois, at Urbana-Champaign.

**Johnson, T. C., Cole, D. M. and Chamberlain, E. J.** (1978) Influence of freezing and thawing on the resilient properties of silt soil beneath an asphalt concrete pavement. USA Cold Regions Research and Engineering Laboratory, CRREL Report 78-23.

**Gilman, G. D.** (1964) Freezing index in New England. USA Cold Regions Research and Engineering Laboratory, Special Report 63.

**Schmidt, R. J.** (1973) The effect of temperature, freeze-thaw and various moisture conditions on the resilient modulus of asphalt-treated mixes. Technical Paper Number 168, Chevron Asphalt Company, San Francisco, Calif.

**Ridgeway, Hallas H.** (1982) Pavement subsurface drainage systems. National Cooperative Highway Research Program Synthesis of Highway Practice 96, Transportation Research Board, National Research Council, Washington, D.C..

## APPENDIX A: CRREL REPETITIVE PLATE BEARING TEST VEHICLE

The basic operation of the RPB vehicle is discussed in the text. Additional design and operating details are given in this appendix. The trailer's gross weight is 30,000 lb. Hydraulic rams, located at each corner of the trailer, are used to lift the trailer wheels off the test surface. Each ram has an 18-in.-square aluminum foot plate that is connected to the ram via a removable pin.

The plate bearing load actuator is a two-chamber pneumatic-hydraulic pressure transformer. To generate the plate bearing load pulse, an air pressure pulse of approximately 80 psi is supplied to the upper chamber. The lower chamber, which is filled with ethylene glycol, converts pressure pulse into a 9,000-lbf pulse. A load cell installed between the lower chamber and the load plate provides a continuous readout of the force transmitted.

A 14.4-cfm two-stage air compressor driven by an electric motor supplies compressed air for the load actuator.

The two-way air control valve actuated by a solenoid generates the compressed air pressure pulse. A solid-state timer regulates the motion of the air control valve. The timer circuit is adjustable to allow variation in the duration of the pressure pulse and the elapsed time between pulses. The duration is continuously adjustable between 0.2 sec and 20 sec. The elapsed time between pulses is also continuously adjustable between 30 repetitions/min to 1 repetition every 3 min. The number of load pulses during the test are automatically recorded.

Flow restrictors with adjustable needle valves installed in the supply and the exhaust line of the load actuator allow adjustment of the rate of pressure rise and pressure release of the load actuator. This adjustment is needed to enable a standardized load pulse to be used at test sites with different response stiffness.

Linear variable differential transformers (LVDTs) are used to monitor the motion of the 12-in.-diam. load plate and the surface deflection basin. The LVDTs (dc-type), which are mounted to an 18-ft reference beam, are positioned with two on the load plate on perpendicular radii and four in the surface deflection basin along a radius common to one on the plate. A strip-chart recorder monitors the LVDT and load cell outputs.

U. S. GOVERNMENT PRINTING OFFICE: 1989-600-057/82075